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REMOVAL OF ALGAE FROM FACULTATIVE POND EFFLUENT

by

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ABSTRACT

Wastewater stabilization ponds have gained popularity as a means of secondary wastewater treatment because of their low cost and simplicity of operation and maintenance, in addition to several other advantages. However, the presence of algae in the effluent from facultative ponds may cause undesirable environmental impacts, such as DO depletion or eutrophication, in water bodies to which this effluent is discharged. Thus, regulations and/or the adverse environmental impacts of effluents containing algae sometimes necessitate reduction or removal of algae from pond effluents.

Many methods have been used for the purpose of removal of algae from wastewater stabilization pond effluent. Upflow rock filtration and coagulation-sedimentation have been investigated in this research.

In the past, mechanisms of removal of algae in rock filters and factors that affect the removal process have not been well explained. Design of these filters has not been related mathematically to the operating variables.

Three pilot rock filters were built for the purpose of this research. The first filter was filled with rock of 1 cm average diameter, the second and third filters were filled with 5 cm and 10 cm average diameter rock, respectively. Five different hydraulic loadings were applied to these filters, with variable influent characteristics and under different environmental conditions.

The results of this research have shown that gravitational settling and hydrodynamic forces are the most important mechanisms affecting removal of algae in upflow rock filters. Also, a method of design of unisize-media upflow rock filters for removal of algae from facultative pond effluent has been established.

The cost of coagulant has been the main disadvantage of the process of coagulation-sedimentation for algae removal from wastewater stabilization pond effluent. In this research, jar tests were carried out to determine the effect of settling time and/or addition of kaolinite or bentonite with the primary coagulant, alum, on the optimum dose of this primary coagulant required for removal of algae from facultative pond effluent.

Both increase in settling time and coagulant aids, namely kaolinite and bentonite, have reduced optimum alum dose. Kaolinite and bentonite, applied as low-cost primary coagulants, were also effective in removing algae from facultative pond effluent.

From the very limited experiments carried out at the end of this research, it was shown that crossflow microfiltration has some potential as a process for algae removal from facultative pond effluent.

A mathematical model has been developed in this work to describe the eutrophic state of King Talal Reservoir in Jordan. The model predicts that even elimination of phosphorus from the effluent of Al-Samra Wastewater Stabilization Ponds will not bring the reservoir into an oligotrophic state.

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SYMBOLS AND ABBREVIATIONS

a, m, n	constants
A	surface area of membrane
A_s	lake surface area
ANSI	American National Standards
APHA	American Public Health Association
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
AWWA	American Water Works Association
b, B	constants
B	particle diffusivity
BOD	biochemical oxygen demand
BOD_t	total biochemical oxygen demand
BOD_s	soluble biochemical oxygen demand
C	concentration of phosphorus in the lake
C.	initial concentration of phosphorous in the lake
C_{pr}	concentration of phosphorus in the precipitate
C_r	concentration of phosphorus in the main river
C_T	concentration of phosphorus in the tributary
C_c	complementary solution
C_p	particular solution
C	concentration of suspended solids at any time at any depth in the filter, lake or filter effluent concentration
C_{in}	filter influent concentration
C_1	constant
C_b	concentration of solids in the bulk solution
C_g	equilibrium gel concentration in the dynamic membrane
C_w	concentration of solids at the wall in the dynamic membrane
CEC	cation exchange capacity
d	day
d_c	diameter of the collector (filter grain)
d_p	diameter of the particle
d_s	settling distance in rock filter
D	diameter of pilot-scale reactor
DO	dissolved oxygen
E	dimensionless group that describes inertial impaction of particle within filter pores
E_v	evaporation from lake
EPA	U. S. Environmental Protection Agency
EMRO	Regional Office for the Eastern Mediterranean
f, i, j	constants
f_s	fraction of particles with settling velocity $v_s<v$
F	filterability number
g	gravitational acceleration
g	gram
G	mean velocity gradient
hr	hour
I	dimensionless group that describes interception
of	particles within filter pores
JTU	Jackson turbidity unit
JD	Jordanian Dinar

J	flux through crossflow microfiltration membrane
k	number of factors, number of subgroups
$\overline{K}$	Boltzman's constant
K	first order decay rate of phosphorous concentration
K_m	back diffusion mass transfer coefficient
kg	kilogram
l	litre
m	metre
mg	milligram
min	minute
n	porosity
NTU	nephelometric turbidity unit
P	precipitation directly on the lake, phosphorus
P_c	fraction of algal species completely removal
P_p	fraction of algal particles with settling velocity v_p P_e Peclet number
pH	the negative log of the concentration of hydrogen ion
q	mass of suspended solids deposited in the filter
q_u	ultimate mass of suspended solids deposited in the filter
Q	outflow
Q_r	inflow from main river
Q_T	inflow from tributaries
r_o	initial removal rate in the filter
rpm	revolution per minute
R	removal efficiency; fraction removal
R_e	Reynolds number
R_m	membrane resistance
R_p	hydraulic resistance
RSS	Royal Scientific Society, Jordan
S	dimensionless group that describe gravitational settling of particles with filter pores
s	second
S_D	rate of phosphorus release for sediment
S_F	dry fallout rate of phosphorus
t	time
T	absolute temperature
TSS	total suspended solids
TP	total phosphorus
Turb	turbidity
U	velocity of fluid at infinite distance relative to filter grain
v	approach velocity
v_c	critical velocity
v_i	interstitial velocity
v_p	velocity of particle relative to fluid
V_s	Stoke's settling velocity of a particle
v_t	tangential velocity of a liquid
V	volume of reactor, volume of filtrate, volume of reservoir
VSS	volatile suspended solids
WHO	World Health Organization
WRC	U. K. Water Research Centre
W (t)	phosphorus input load

$\bar{W}$	constant phosphorus input load
X	height of filter
x_{ji}	variable j at level i
Y	estimate of response parameter
Y_{all}	estimate of response parameter using all the data
Y_j	response parameter by omitting the jth group of the data
Y_{ji}	response parameter j at level i
Y_{j*}	pseudo value of Y_j
α	collision efficiency factor
α, β	constants in
β_p	packing constant
β_o, β_1	constants
β_2, β_{11}	constants
β_{22}, β_{12}	constants
γ, δ	constants
Δ	the fraction of solids retained in a filter layer one grain diameter thick
ΔP	pressure drop between upstream and filtrate
P	power delivered to a fluid
ϵ	porosity of filter
η_c	single collector efficiency factor
$(\eta_c)_D$	single collector efficiency factor for diffusion
$(\eta_c)_I$	single collector efficiency factor for gravity sedimentation
θ	detention time in rock filter
λ	filter coefficient
λ°	initial filters coefficient
$(\lambda_\circ)_{all}$	estimate of $\lambda_\circ$ using all the data
$(\lambda_\circ)_1$	estimate of $\lambda_\circ$ omitting group 1
$(\lambda_\circ)_2$	estimate of $\lambda_\circ$ omitting group 2
$(\lambda_\circ)_3$	estimate of $\lambda_\circ$ omitting group 3
μ	dynamic viscosity
π	ratio of circumference to diameter of a circle
ρ_p, ρ_s	density of solids
ρ	density of fluid
ρ_w	density of water
σ	specific deposit in mass/volume
τ	dummy variable
Ω	specific deposit in volume/volume
Ω_u	ultimate specific deposit in volume/volume

CHAPTER 1

Introduction

Wastewater stabilization ponds have become the most popular form of secondary treatment in developing countries. In developed countries, they are mostly used for small communities and industrial wastewater treatment. Ponds can be operated and maintained at low cost. Less foreign currency is required for their construction and operation. Also, ponds are effective in removal of helminth eggs. This feature of ponds is a very important advantage from the health point of view. However, the presence of algae in the effluent makes its quality inferior in terms of TSS and BOD.

The long term average TSS of facultative pond effluent is 100 mg/l with algae forming the majority (Gloyna, 1971). Tapp et al. (1976) reported an average of 120 mg/l, while Parker (1976) reported a range of 75-150 mg/l. It has been reported that the concentration of suspended solids in facultative pond effluent may exceed 100 mg/l (Middlebrooks et al., 1982; Reed et al., 1988).

Environmental conditions and/or regulations, sometimes require the removal of algae from pond effluent. Many methods have been investigated for such purpose.

1.1 The Need to Remove Algae

Algae may be harvested for their fertilizing value or for their protein content as an animal or fish food (McGarry et al., 1972; McGarry, 1974). However, sometimes algae are required to be removed because of their detrimental effect on the environment. McKinney (1976) pointed out that pond effluent organic carbon may exceed that of the influent to ponds. Thus, if the aim of ponds is to remove BOD from the influent, then algae should be removed from the effluent (Oswald et al., 1970). McKinney (1974) and EPA (1974) considered the removal of algae from ponds effluent a condition for successful operation and production of good quality effluent . Parker et al. (1973) found

that most of the long term BOD was associated with algae in the effluent of the ponds in Stockton, California. Removal of algae will make the effluent meet the limitations on BOD exerted by wastewater on receiving water. It was required that the waste discharge should not cause the dissolved oxygen of the receiving waters to fall below 5.0 mg/l at any time.

The environmental adverse effects of algae on receiving bodies of water mainly include BOD exertion and depletion of oxygen, nutrient supply and eutrophication (King et al., 1970; Parker et al., 1973; King, 1976).

Pond effluent has caused oxygen depletion and fish kills in the San Joaqum River (Bain et al., 1970). Experimental studies carried out at the Asian Institute of Technology suggested that if the effluent from an oxidation pond system treating municipal sewage has an algal concentration greater than $2x10^5$ cells per ml, the stream to which the effluent is discharged could turn anaerobic before sunrise, even though it may be saturated with oxygen during the day (Pescod, 1974). Middleton and Bunch (1970) recommended removal of algae from pond effluent since 1 mg/l of dead algae is equivalent to 1.5 mg/l BOD_5. Bare et al.(1975) found that 1.11 mg of O_2 are needed to oxidize 1 mg of dry algae, obtained from laboratory studies, to CO_2 and water. For algae obtained from field studies, 0.81 were required to oxidize 1 mg of algae. Varma and Digiano (9168) found that o.67 mg of oxygen, measured as COD, is required for each milligram of algae. Friedman et al. (1977) found that 1.19 mg COD has the equivalent of 1 mg algae for both laboratory and field experiments. Oswald and Ramani (1976) reported that 1.4 mg/l is the oxygen demand of 1 mg dry weight of algae in one litre suspension. Theoretical calculation of oxygen demand created by algae destruction suggests that 1.58 of O_2 are required to oxidize one mg of dry algae to CO_2 and water (Ward and King, 1976).

In addition to the dead algae demand for O_2, live algae excrete organic compounds that are equivalent to 37-148 mg/l of dissolved COD (Ward and King, 1976). Palmer (1980) reported that some species of *Chlamydomonas* and *Scenedesmus* excrete up to 40% of the oxidizable soluble material formed during algal photosynthesis. Also, algae eventually will die, exerting an oxygen demand on the receiving streams (Shindala and Stewart, 1971). Tests carried out by

Shindala and Stewart (1971) revealed that algae contributed 50% of BOD_5, COD and Nitrogen in the samples. Less than 25% was the share of algae in phosphates.

Although the presence of nutrients in wastewater treatment plant discharges may be desirable for their fertilizer value, if the effluent is used in agriculture it should be noted that the continuous application of nutrients with irrigation water has some adverse effects. It was found that if the concentration of nutrients is greater or less than optimum, crop yield may be reduced (Ingestad and Lund, 1979; Casanova, 1982; Jones, 1982).

It was observed that high nitrogen concentration in irrigation water supply, especially if applied towards maturity, may result in excessive vegetative growth, or lodging and delay of crop maturity and affects the quality of fruits and productivity. Problems appear if nitrogen concentration is above 30 mg/l (Ayers and Westcot,1976; Pettygrove and Asano, 1984; Pescod, 1992).

Nitrogen may accumulate in crops (Cowan and Jhonson, 1984). However, it was found that the nitrate content in lettuce decreased when N concentration was decreased in a hydroponic system four days or more before harvest (Alt and Strüwe, 1982).

The presence of nutrients in the effluent of wastewater treatment plants has been considered the main cause of eutrophication in water bodies to which this effluent is discharged. Many attempts to control eutrophication in lakes and rivers have concentrated on removal of nutrients from wastewater treatment plant effluents, or diversion of these discharges from the eutrophic bodies of water (Porcella, 1972; Canale, 1979).

Wastewater stabilization ponds, in general, have been found unable to remove N and P (Barsom and Ryckman, 1970). Hence, removal of algae from pond effluent will reduce these nutrients (Middleton and Bunch, 1970; Al-Layla and Middlebrooks, 1975).

Other problems that may be associated with the presence of algae include clogging of sand filters and shortening of filter runs. It was recommended that algae should be removed prior to treatment of surface water in slow sand filters (Visscher et al., 1987). Algae may adsorb heavy metals. Blue green algae may

produce substances that may cause digestive disturbances, neuromuscular disfunction or even death. Also, green algae such as chlorella may cause human gastroenteritis or skin allergy (Lewis, 1979).

In Jordan, Al-Samra Wastewater Stabilization Ponds effluent has been blamed for eutrophication in King Talal Reservoir (Salameh et al., 1987). Other specific problems related to Al-Samra Wastewater Stabilization Ponds in Jordan, include: clogging of the drip irrigation system in the agricultural project within the premises of the plant. Clogging by algae made the system ineffective and had to be removed and replaced by furrow irrigation which is more water consumptive in a country where scarcity of water makes it necessary to save every possible drop. Another problem is related to chlorination of the effluent. The effluent of the ponds has to be chlorinated according to the requirements of the Ministry of Health. The presence of algae may interfere with disinfection, increase chlorine dose, produce odour and even harmful chlorinated hydrocarbons (Hom, 1972; Middleton and Bunch, 1970; Middlebrooks et al., 1982).

The effluent from Al-Samra ponds runs in a dry wadi and during floods, stagnant impoundments of water are created forming a good habitat for mosquito and fly breeding, and resulting in odour nuisance. Complaints from local people have been reported very often to the Water Authority of Jordan because of these swamps. In the next Winter season, sediments in these impoundments are scoured and driven to King Talal Reservoir, causing significant deoxygenation of its water.

Finally, the TSS average concentration in the effluent ranged between 100 and 240 mg/l in the years 1986-1990 (RSS, 1991). This is very much higher than the maximum limit of 50 mg/l set by the Modified Wastewater Quality Standards in Jordan for discharges to wadis, rivers or reservoirs. It is even higher than 100 mg/l, the maximum limit for reuse of effluent in irrigation.

1.2 Methods Utilized in Algae Removal from Wastewater Stabilization Pond Effluent

Depending on several factors such as: algae content in effluent, end use of effluent, effect of effluent on the receiving body of water , and requirements of local regulations and ordinances, algae may have to be removed from the effluent of facultative and maturation wastewater stabilization ponds. Several, methods have been reported in the literature for removal of algae from pond effluents (Middlebrooks et al., 1974; WHO/EMRO, 1981):

1-In pond or effluent chemical treatment
2-Integrated ponding
3-Series ponds with intermediate chlorination
4-Dissolved air flotation
5-Autoflotation
6-Filtration
7-Microstraining
8-Centrifugation
9-Hyacinths and duckweed
10-Others

1.2.1 In Pond or Effluent Chemical Treatment

Instead of removal of algae from pond effluent, research has been carried out to upgrade the effluent quality by chemical precipitation in the pond. Chemicals have been added to the influent of the pond and the pond has served as a sedimentation tank. This mode of operation is more simple and has the advantage of no addition of structures, such as flocculation and sedimentation tanks, but ponds will suffer from heavy accumulation of sludge (Balmér and Vik, 1978; Middlebrooks et al., 1974).

However, chemical addition to the influent of the pond did not prevent the formation of algal mats inside the pond (Balmér and Vik,1978). Addition of

chemicals to the influent in a full treatment unit is also faced with the difficulty of controlling coagulant dose because of variation of influent flow and characteristics. In addition to that, better removal of TSS, TKN, TP and COD was achieved by use of tertiary treatment of the effluent compared with addition of coagulant to raw wastewater or spreading it inside the pond (ibid). Also, overdose of alum to the pond has flocculated algae and induced a decrease in oxygen concentration. Shindala and Stewart (1971) found that significant removal of BOD, COD, N, P and algae could be achieved by tertiary treatment of pond effluent.

In other secondary treatment systems, it was found that addition of aluminium salts to the primary units or biological reactors removed phosphorus but did not significantly reduce the concentration of TSS, BOD, COD (Dobolyi, 1973). Many work in support of these findings were reported by Linstedt et al. (1974). On the other hand, tertiary treatment by alum achieved phosphorus removal accompanied with improvement in overall quality of the effluent, including TSS, BOD and coliform count (Malhotra et al., 1964; Stukenberg, 1971; Williams and Malhotra, 1974). Jenkins et al. (1970) recommended tertiary treatment in preference to primary or combined biological chemical treatment if better removal of phosphorus is required.

Other researchers have investigated in-pond algae removal utilizing intermittent discharge or isolation ponds (Koopman et al.,1979). The effluent from facultative ponds was isolated in a separate lagoon which was drawn and filled over a cycle. The duration of the cycle was 22 days from the start of filling to the end of discharge. Filling took 2 days, and emptying took 3-5 days. The system was found to achieve an average of 67% removal of algae from facultative pond effluents.

Koopman et al. (1979) faced many difficulties with such operation. Wind induced resuspension of settled solids and blue green algae developed. For more removal they needed another step, such as intermittent sand filtration.

It must be noted that in-pond algal precipitation requires a large storage volume, especially when the cycle of fill and discharge may extend to six months if undesirable conditions for discharge occur, such as the development of algal

blooms during summer or the growth of blue green algae. Odour may also develop in the system (Ford and Tischler, 1976) .

This system is not suitable if the effluent is to be used for irrigation; the effluent will be available in Winter when it is not needed and it will be held in Summer when plants need to be irrigated. In Jordan, this is considered a main disadvantage because pond effluent is totally used for irrigation.

1.2.2 Integrated Ponding

Wastewater stabilization ponds are designed such that each pond is optimized for a special requirement in the overall treatment process. When these ponds are placed in series the entire system is referred to as integrated ponding (Koopman et al., 1979). With proper design and under favourable environmental conditions, either primary treatment or a facultative pond preceding a high rate pond, followed by a short detention time settling pond will result in removal of 70% or more of the algae in the high rate pond effluent without chemical additives or special mechanical equipment. Mechanisms involved are carbon dioxide uptake and alkalinity precipitation which assists the settleability of algae. Mixing in the high rate pond also enhances flocculation (ibid).

Disadvantages include: the need for larger area (Golueke and Oswald,1965), the effluent may need further polishing by rapid sand filters, the requirement of a great degree of operation attention, a separate basin is required for settling, very quiescent conditions are required, autoflocculation conditions must be achieved, and the method is related to proper environmental conditions (Middlebrooks et al., 1974; Koopman et al., 1979).

1.2.3 Series Ponds with Intermediate Chlorination

Parker (1976) reported that the addition of chlorine to pond water will kill algae and cause quicker settling. The flocculation effect of chlorine is thought to be due to the release of cellular metabolites that may serve as algal coagulants (Echelberger et al.,1971). However, the use of chlorine for removal of algae has

the disadvantage of increasing soluble BOD (Echelberger et al.,1971; Hom,1972). Also it may cause the formation of THM's which are suspected to be carcinogenic.

Optimum dose is not easy to define. It is affected by the concentration of algae, which is variable. An overdose will destroy the algal population, which disrupts the treatment process itself. A lower dose may not be effective in control of algae.

1.2.4 Dissolved Air Flotation

Dissolved air flotation (DAF) utilizes Henry's law to obtain solubility of gas in a liquid.The saturation of air in water is directly proportional to the pressure applied and inversely proportional to the temperature. Liquid can be saturated with air under pressure and when the pressure is released ,under proper hydraulic conditions, the air comes out of solution forming bubbles surrounding finite nuclei. Coagulants are added to form the basic flocs in which air is entrapped. The combined air-solid mass has a specific gravity less than the liquid and floats to the surface(Snider, 1976; Stone et al., 1976).

The required alum dose reported in the literature varies from 50 to 400 mg/l, depending on water characteristics (Snider, 1976).

McGarry and Tongkasame (1971) tested several coagulants, including aluminum sulphate, lime, ferrous and ferric sulphate, ferric chloride and fifty polyelectrolytes.They found that alum was the most feasible inorganic coagulant.

The technique of DAF achieved removal of TSS in the range of 70% to 90% (McGarry, 1974; Bare et al., 1975; Stone et al., 1976). High hydraulic loading is quite feasible, which gives DAF an advantage that smaller tanks are required by comparison with coagulation-sedimentation. Flotation can be accomplished in shallow tanks with a hydraulic retention time of 7 to 20 minutes compared with 3 to 4 hours in deep sedimentation tanks (Parker,1976).

However, 3 to 4 hours is not a long detention time compared with the detention time of wastewater stabilization ponds. So, the difference in retention time between flotation and sedimentation can not be considered as a great

advantage for dissolved air flotation as a method of removal of algae from wastewater stabilization pond effluent.

Although some removal of dissolved BOD is achieved by DAF, the degree of removal is unpredictable (Snider, 1976). Also, the method requires skilled supervision and dos not satisfy the basic requirements of simplicity and ease of operation which are characteristic of ponds (Middlebrooks et al., 1974; Tapp et al., 1976).

The process is relatively expensive and has not generally proved to be cost effective (Middlebrooks et al., 1974; Tapp et al., 1976; Kormanik and Cravens, 1979).

In addition, large volume of sludge is produced because of low solids concentration in the skimmings. This introduces an additional operational problem.

1.2.5 Autoflotation

Algae supersaturated with oxygen release part of that oxygen when entering the flotation chamber.This causes algae to float to the surface (McGarry and Tongkasame, 1971). Super-saturation may be released by aeration, carbon dioxide addition, and mixing (Van Vuuren et al.,1965).

Parker (1976) concluded that the system needs more flotation tanks (double or more) than the pressurised dissolved air flotation technique because the system is an intermittent process.

The system cannot be relied upon alone because supply of O_2 by algae is not continuous (Caldwell et al., 1973). In addition, in order to be effective DO must reach 14 mg/l, which would not be attained during the night and early morning (Parker et al., 1973).

1.2.6 Filtration

Filtration techniques employed include rapid uni- and multi-media filtration, slow sand filtration, intermittent sand filtration, roughing filtration, and rock filtration.

(I)Rapid Filtration

The wastewater is passed through one or more layers of media such as coal, sand or anthracite. Suspended solids are removed by different mechanisms, such as straining, inertial impaction, thermal diffusion, gravitational settling and others. A summary of these mechanisms may be found in the work of Cleasby (1972).

Polishing filtration following a coagulation -clarification step has been described as practical and it could produce an effluent of less than 3 JTU and 10 mg/l or less of TSS (Stone et al., 1976). The process was considered economically feasible for the removal of algal cells from lagoon effluents. Coagulants may be added such that filters may be used as chemical treatment units (Middlebrooks et al., 1974).

In spite of these advantages, Parker (1976) concluded that removal of algae has been generally inefficient using rapid sand filters without addition of coagulants. Investigations reported by Ford and Tischler (1976) revealed that discrete particles of algae penetrate deeply in the filter and removal was only marginally successful. Also, filtration cycles were too short, reaching 30 minutes at a filtration rate of 3 m/h with influent containing 75 mg/l of total suspended solids. Another drawback of employing rapid sand filters for algae removal is slime growth throughout the filter media, which makes cleaning difficult. In addition to that, equipment is needed for backwashing.

(II) Slow Sand Filters

A slow sand filter consists of a watertight basin containing a layer of sand 1-1.5 m thick supported on a layer of gravel. The gravel is underlaid by a system of open joint underdrains. The common effective size of sand is 0.35 mm and a typical uniformity coefficient is about 1.75. The filter is operated with a water depth 1-1.5 m above the sand surface (Cleasby, 1972).

Slow sand filters act as strainers. They normally accumulate most of the impurities at the sand surface and in the top 5 cm or so of sand. Large microbic populations multiply in the accumulating organic matter, nutrients, or schmutzdeke (Fair et al., 1968).

Slow sand filters offer cost savings due to elimination of coagulation but this is possible only if turbidity is less than 50 NTU; otherwise, turbidity should be reduced by coagulation prior to slow sand filtration. Other advantages include: the possibility of capturing small species of algae; backwashing equipment is not needed and cleaning may be accomplished manually. However, this may be considered a disadvantage where labour cost is high. Less skilled supervision is required compared with rapid sand filters (Huisman and Wood, 1974; Parmasivam et al., 1981).

The disadvantages of slow sand filters are summarized as follows, based on the work of Cleasby (1972), Huisman and Wood (1974), Thanh and Pescod (1974):

Disadvantages include the deep penetration of fine suspended solids, such as clay or small algal species. If these fine particles are present in large amounts, cleaning of the filter cannot be achieved by normal scraping methods. Clogging may also take place due to precipitation of bicarbonate alkalinity as $CaCO_3$ at high pH values , which result from algal activity in the supernatant.

Slow sand filters are operated at low hydraulic rates, which require large surface area and, consequently, a high capital cost when land is expensive.

Anaerobic conditions may develop in these filters, especially during darkness or at low temperatures, due to decomposition of algae which will result

in production of soluble BOD, deficiency of oxygen, production of odour and release of nutrients.

In addition, only low turbidity and low algal concentration in the influent is suitable for treatment by slow sand filters.

Efficiency will be adversely affected at low temperature and structural precautions against freezing should be provided where very low temperatures are experienced.

(III) Intermittent Sand Filtration

An intermittent sand filter is the same as a slow sand filter but pond effluent is applied on a periodic or intermittent basis to the filter bed (Middlebrooks et al., 1982). Fair et al.(1968) stated that water may be applied once or more each day during 7 to 20 minutes.

Laboratory, pilot and field investigations were carried out at Utah State University on removing algae from wastewater stabilization pond effluent utilizing intermittent sand filtration (Harris et al., 1977; Reynolds et al., 1976; Reynolds et al., 1979; Tupyi et al.,1979; Middlebrooks et al., 1982). A hydraulic loading rate of 0.19-1.13 m3/m2.d was found to have little effect on TSS, VSS, and BOD in the laboratory portion of the investigation. A slight decrease in removal of these parameters occurred when hydraulic loading was increased in the field studies. The researchers concluded that intermittent sand filtration can produce an effluent with BOD < 10 mg/l, TSS < 10 mg/l, VSS < 5 mg/l and winter operation of the filters did not create any serious problems. The work was done with effluents containing an average of 26.1 mg/l and a maximum of 72.1 mg/l of TSS concentration.

The advantages and disadvantages of intermittent sand filters are the same as those of slow sand filters but it should be noted that: intermittent sand filters may not turn anaerobic; large storage volume is needed in this process; and the effluent quality may be reduced due to unstable working conditions.

Parmasivam et al. (1981) reported deterioration in the bacteriological quality of effluent after a short time from start up because of unsteady conditions of the intermittent loading and discharge process.

(IV) Roughing Filters

Roughing filters use much larger media than either slow or rapid filters. Slow sand filters utilize 0.15 - 0.35 mm diameter media; rapid sand filters are usually filled with sand of 0.4 - 0.7 mm diameter; roughing filters use media > 2.0 mm diameter (Schulz and Okun, 1984). Wegelin (1988) classified roughing filter material size in the range 4-25 mm operating at a filtration rate of 0.3-1.5 m/h.

Roughing filters are often used before slow sand filters because of their effectiveness in removing suspended solids (Schulz and Okun, 1984).

McGarry (1974) used roughing filters at the Asian Institute of Technology to capture algae escaping dissolved air flotation and as a step before rapid sand filters. He found that roughing filters extended rapid filter runs between backwashes.

Roughing filters can be operated in the vertical or horizontal flow mode. They may be packed with materials such as shredded coconut fibres or burnt rice husks (Shulz and Okun, 1984). Virtually, depending on availability and economy, any inert, clean, insoluble and mechanically resistant material can be used in packing roughing filters. Examples include broken burnt bricks in the Sudan and palm fibre in Java, Indonesia (Wegelin, 1988).

(V) Rock Filters

Rock filters may be used as a roughing tool preceding other removal processes, but they usually serve as the prime process for algae removal from wastewater stabilization pond effluent (Metcalf and Eddy, 1979).

Uncertainty exists about the effect of influent suspended solids on rock filter performance (Parker, 1976). The performance of rock filters in the Winter

may be lower than in Summer. O'Brien (1976) recommended that lower hydraulic loadings be applied to rock filters in Winter. The clogging and service life of these filters determine their feasibility as one of the alternatives for algae removal (Rich, 1988). Cleaning of horizontal flow rock filters may require removal of the whole packing material.

Rock filters operated for algae removal from pond effluent achieved 30 mg/l monthly average of BOD and TSS concentration in the effluent (O'Brien, 1976). Because of its simplicity and ease of operation and maintenance, the rock filtration process is compatible with the simple, low cost operation and maintenance of ponds (Swanson and Williamson, 1980; Middlebrooks, 1988).

1.2.7 Microstraining

A microstrainer is a low speed (4-7 rpm) rotating drum filter operating under gravity conditions. Fabrics of finely woven stainless steel are fitted on the drum with mesh openings are usually in the range of 23 to 60 microns (Middlebrooks et al., 1974). Stainless steel microstrainers are subject to fouling and corrosion (Oswald and Golueke, 1968). The development of non corroding plastic screens may prove more suitable (Koopman et al., 1979).

Middlebrooks et al. (1974) reported successful application of microstraining for removal of algae from water supplies.

Golueke and Oswald (1965) conducted pilot plant studies at flow rates of 187-378 l/min and obtained an extremely low degree of algae removal by the machine, even with addition of filter aid, decrease in flow rate, and slowing of rotational speed of the filter drum from 30 to 20 and 10 rpm.

Parker (1976) concluded that planktonic algae are not removed efficiently by microstrainers because of their size (usually less than 20 microns, while the smallest size microstrainer opening commercially available is 23 microns) .

In extensive full scale microscreening studies carried out by Kormanik and Cravens (1979) at seven sites in the U.S.A., it was shown that microstraining by a one micron polyester fabric is a viable process alternative for the removal of algae. Influent TSS ranged between 26 and 126 mg/l with TSS in the effluent in

the range of 9 - 22 mg/l. No operational problems were encountered at any of the sites tested.

Similar successful experiments were carried out by Shelef et al. (1972) for removal of algae from wastewater stabilization pond effluent. Shelef et al. (1972) used ultrafiltration-microfiltration with mesh size 0.2 microns and obtained 98.8% removal of TSS. The predominant species of algae were *Chlorella*, *Scenedesumus* (< 20 μm) and *Euglena* (15 - 30 μm).

Koopman et al. (1979) argued that the parameters affecting operation of microscreens for algae removal from pond effluent are the particle size distribution and concentration. Particle size depends on the algal species that populate the ponds, their colonial aggregation and flocculation behaviour. High flocs result in high algae removal. A relatively high concentration also helps algae removal as it results in a formation of "a precoat" that helps to trap materials which are smaller than would normally be retained.

High rate pond systems allow both achievement of relatively high algal concentrations and some control over the algae population, which could be used to selectively cultivate larger filamentous or colonial algae types readily removable by microstrainers (Koopman et al., 1979). However, they conducted research depending on this concept but could not succeed in maintaining colonial algae continuously. Natural upsets in algal populations led to deterioration of effluent quality.

Generally, microstraining utilizing 1 micron polyester fabric effectively removes algae and achieves 30/30 effluent (Kormanic and Cravens, 1979). Microstraining has been shown to require only a minimum operating experience and low operating expenses (Middlebrooks et al., 1974; Kormanik and Cravens, 1979).

The drawbacks of microfiltration include difficulty in maintaining a consistent effluent quality during fluctuation in influent loading. This problem may be partially overcome by varying the speed of rotation (Middlebrooks et al., 1974). Build-up of bacterial and algal slime on the microfabric requires cleaning and results in low flow rates through the system (Middlebrooks et al., 1974; Ford and Tischler, 1976). Microstraining has proved unsuccessful in algae removal if

usual size openings of 23 microns are used (Golueke and Oswald, 1965; Parker, 1976). Ford and Tischler (1976) concluded that microstraining is highly expensive, uneconomical and complex for routine operation.

1.2.8 Centrifugation

Field scale experiments conducted on algae removal by centrifugation resulted in 60%-70% suspended solids removal at a feed ate of 1460 l/min, and 80%-90% removal at a rate of 378 l/min, regardless of the rotor speed in the range tested (100-350 rpm)(Golueke and Oswald, 1965). The investigators concluded that centrifugation consumes high energy with a high capital, operation and maintenance cost of the equipment. Middlebrooks et al.(1974) added that the process needs power and skilled labour and continuous maintenance.

1.2.9 Hyacinths and Duckweeds

Macrophytes such as water hyacinths and duckweeds have been used as a means of wastewater treatment. High fat and protein content make them an attractive food source for animal and poultry. Duckweeds are easier to harvest than hyacinths.

Hausser (1984) tested hyacinths for removal of ammonia from facultative wastewater stabilization ponds. Removal of 70% of ammonia was obtained. Effluent DO decreased in late winter and spring due to the death of hyacinths in Winter.

Zirschky and Reed (1988) showed that suspended solids in lagoon effluent was reduced by introducing duckweeds to ponds.

At the Asian Institute of Technology (AIT), hyacinths were implanted to remove algae from a facultative pond receiving effluent from an anaerobic pond. Ponds containing hyacinths had less suspended solids in their effluent compared with a normal facultative pond (Orth and Sapkota, 1988). The conclusion was that the water hyacinth pond exhibited in all measured parameters (TSS, COD,

TKN, TP) a much higher quality than the facultative pond with, the only exception being the DO concentration. The improvement was particularly important with respect to TSS concentration.

The disadvantages of hyacinths and duckweeds can be summarized as follows (Serfling and Alsten, 1979; Reed et al., 1988; Zirschky and Reed, 1988):

- Water hyacinths cannot withstand cold temperatures and die under winter conditions, whereas duckweed suffer if pH goes above 10.5, or if temperature is as high as 35-40°C.

- Duckweeds are easily blown by winds and need to be confined to keep them distributed over the pond. Because of the lack of surface aeration, anaerobic conditions may result. Frequent harvesting is required to obtain satisfactory performance. Duckweed may be transferred to other cells of the lagoon system, creating a mat that limits oxygen transfer which could affect the performance of the whole system. Fly and mosquito breeding is a problem in floating macrophyte ponds.

- Also, pathogen die off is poor in these ponds because of light shading and lower pH and DO by comparison with maturation ponds (Pescod, 1992).

- In many locations, water hyacinths are considered as a noxious weed, and environmental concern must be evaluated before their use.

- Water hyacinths increase evapotranspiration water losses by as much as 2 to 7 times normal rates.

1.2.10 Other Methods

Several other methods have been investigated for control or removal of algae from pond effluent. Methods investigated include passage through a charged field, ion exchange, sonic vibration, biological harvesting and land application.

Golueke and Oswald (1965) investigated the possibility of using the negative charge of algae to cause them to migrate to the cathode of an electrolytic cell as an algal suspension is passed through the cell. Excellent separation occurred when aluminium or copper electrodes were used because an

excellent floc was formed by the release of copper and/or aluminium and the subsequent formation of copper and/or aluminium hydroxide. No removal was noticed when carbon electrodes were used.

The process has not been tried on a full scale operation but it seems to be power intensive and requires skilled operation.

In the studies of Golueke and Oswald (1965) they found that algae could be removed through a column of either strong or weak cation exchange resin. As the columns lost the cation exchange capacity, algae also lost their tendency to coagulate. They concluded that harvesting of algae by ion exchange is an uneconomical process. Also, the columns had to be washed more frequently than would have been necessary for deionization alone because of algae clogging the column.

Algal cultures have been exposed to ultrasonic waves, starting from 15,000 cycles. Instead of aggregating, algal cells were dispersed effectively at all the frequencies tried (Golueke and Oswald, 1965).

Reid (1976) suggested that the use of aquaculture may solve the problem of high suspended solids in the effluent of ponds. However, Middlebrooks et al. (1974) reported the process of removing algae by fish and /or high plant consuming vertebrates has been to a large extent unsuccessful. Fish excrete their own faecal matter and add to the BOD of the system. These organisms cannot withstand the varying conditions of temperature, flow, and oxygen. The process is affected by the level of oxygen, toxins and ammonia. In fact, oxygen may be depleted due to the high respiration rate of phytoplankton at night (Pescod, 1992).

Wastewater effluent may be applied to the land as an irrigation water or for the purpose of allowing the water to percolate into the soil. During passage through the soil different physicochemical and biological processes act together to purify the percolating water. In fact, this is more a utilization disposal method than a technique for algae removal.

1.3 Summary

Since wastewater stabilization ponds are characterized by simplicity and low cost of maintenance and operation, then any process added to the system should preferably be in harmony with such features. Based on the quick review in this Chapter, of the different methods utilized for algae removal the following systems may be considered to match with the criteria of simplicity and low cost of operation and maintenance of wastewater stabilization ponds: water hyacinths and duckweed, biological harvesting, intermittent discharge lagoons, an integrated pond system, land application, slow sand filtration, rock filters and, finally, coagulation-sedimentation if cheap coagulants are found or doses are optimized.

Floating macrophytes need continuous harvesting and their performance is affected by low and high temperature, which restricts their use. Besides, such plants themselves should be controlled because in many areas they are considered environmentally unacceptable.

Biological harvesting has not proven to be consistently successful in algae removal. The key to the success of the operation may be the ability to control variations in temperature, pH, DO, ammonia nitrogen and other toxic elements. Most of the time this is practically infeasible.

The operation of intermittent discharge lagoons depends on diurnal and seasonal variations of weather conditions and wastewater characteristics; their performance is not consistent as a function of time. This may result in long times between the emptying and refill cycles, which reduces to the fact that vast areas of land would be required.

What applies to intermittent discharge lagoons also applies to integrated pond systems in terms of inconsistent functioning and dependence on environmental and wastewater characteristics.

Land application, as mentioned earlier, cannot be categorized as an algae removal system but rather a utilization-disposal method. Also it needs a large area and depends on location, soil characteristics, topography, effluent

characteristics, and depth of water table. Its application is only limited to specific places.

Slow sand filters require large areas. The possibility of deep penetration by algae in such filter makes cleaning infeasible.

Rock filters have proven successful in their ability to remove algae from pond effluent. They offer a simple and economic means of algae removal. However, clogging of filters of small size rock and uncertainty about the service life of the filter are the main problems facing this technique. These drawbacks have primarily occurred in horizontal flow mode. However, it can be reasonably assumed that vertical upflow operation will eliminate the problem of cleaning, since backwashing is more effective and can be done more easily in this mode compared with horizontal flow rock filters.

Since sometimes high levels of removal of algae are required, coagulation-sedimentation offers a worthwhile solution. The cost of the process makes its use with ponds infeasible. However, if cost can be lowered, the process may be considered a good alternative.

1.4 Objectives

Although many rock filters have been constructed for the purpose of removal of algae from pond effluent, the design of these filters is still preliminary and is not based on the relationship between operating variables and the performance of the filter. Reported inconsistencies on the performance of rock filters reveals the lack of knowledge about the mechanisms involved in the process of removal.

Also, the relation between filter effluent quality and different variables, such as rock size, hydraulic loading and concentration of total suspended solids in the influent, has not yet been defined.

This research was initiated to study removal of algae by upflow rock filters from facultative pond effluent. The general aim was to establish a design basis for these filters.

Decreasing the dosage of coagulants required for algae removal and employing cheap coagulants, such as clay, were also principal objectives of this research.

Since high quality effluent is sometimes required, it was thought that crossflow filtration might be a useful new technology for algae removal.

These were the general aims of the research and the specific objectives are summarized as follows:

1. To investigate upflow rock filters as a potential process for algae removal from facultative pond effluent.

2. To identify the most effective mechanisms of removal of algae in the pores of upflow rock filters and to determine the effect of these mechanisms on filter performance.

3. To determine the relationship between filter performance and variables such as hydraulic loading, total suspended solids concentration of the influent and rock size. Also, to investigate the interaction among these variables.

4. To formulate the relation between these variables into a mathematical model such that it could be adopted as a method of engineering design of rock filters.

5. To determine the capability of alum aided by kaolinite or bentonite for removal of algae from pond effluent.

6. To determine the effect of these coagulant aids on reduction of alum dose.

7. To investigate the possibility of reduction of alum dose by increasing settling time in the settling basin.

8. To determine whether kaolinite or bentonite are potentially viable primary coagulants for removal of algae from pond effluent.

9. To evaluate the capability of rock filtration or coagulation-sedimentation for removal of other constituents of pond effluents, such as BOD, nitrogen and phosphorus.

10. To investigate the possibility of removing algae from facultative pond effluent employing crossflow filtration.

11. To construct a model that may be useful in the decision making process for control of eutrophication in King Talal Reservoir in Jordan.

CHAPTER 2

Removal of Algae by Rock Filter

Filtration is a physical, chemical and, sometimes, biological process by which water or wastewater is passed through a process media to remove suspended solids and colloidal impurities which might be present in water or wastewater. Suspended solids may be the result of previous processes, such as algae in wastewater stabilization pond effluent, precipitated hardness, precipitated iron and manganese, biological floc after sedimentation of a secondary treated wastewater, flocs due to coagulation and, finally, dewatering of sludge (ASCE et al., 1971; Schulz and Okun, 1984).

2.1 Classification of Filters

Filters may be classified, according to the media used, into sand filters, rock filters, roughing filters, anthracite coal filters and diatomaceous earth filters. Perlite and activated carbon are also used in filters. These materials may be used alone, in single media filtration or multiple media filtration can also employed (Cleasby, 1972).

The rate of filtration and media size distinguish rapid sand filters, slow sand filters and roughing filters. The advantages and disadvantages of the use of these filters for removal of algae have been reviewed in Chapter 1.

Another way of classification of filters is according to the driving force, for example, gravity and pressure filters. According to the mode of operation, filtration may be classified into constant-pressure filtration, constant-rate filtration and variable declining-rate filtration (ASCE et al., 1971; Cleasby, 1972; Metcalf and Eddy, 1991).

The direction of flow through a filter bed is also used to distinguish filter types: horizontal flow, down flow, upflow, biflow and also crossflow filtration are all different classes of filtration processes (Ives,1961; Metcalf and Eddy, 1991).

A review of different kinds of filters may be found in the work of the Task Committee on Design of Wastewater Filtration Facilities (ASCE, 1986).

Upflow rock filtration for algae removal has only been tested on a lab scale unit. Limestone rock, 2.5-5 cm diameter was filled in a pipe of 10 cm internal diameter to a height of about one metre. This study showed that rock filters offer a potential method that could be employed for algae removal (O'Brien et al., 1973).

Reported field-scale rock filters, which are utilized for algae removal from wastewater stabilization pond effluent, have all been designed and operated in a horizontal flow mode (see, for example, O'Brien, 1976; O'Brien and McKinney, 1979; Middlebrooks et al., 1982).

Horizontal flow roughing filters have the advantages of low construction cost and the possibility of building any required length of filter. Vertical flow filters are limited by the height that can be practically constructed and their construction cost is higher (Schulz and Okun, 1984).

However, horizontal flow rock filters suffer a main disadvantage that their storage capacity and, consequently, their serviceable life is yet uncertain and speculative (Tap et al., 1976). In fact, field-scale research on rock filters of 3 cm-0.47 cm media size at Eudora, Kansas resulted in them clogging and being taken out of operation only 11 months after commissioning (O'Brien, 1975,1976).

If blockage occurs, regeneration of horizontal flow filters may require removal of the filter media. Schulz and Okun (1984) stated that manual cleaning is required for horizontal flow roughing filters. This may be plausible for small filter units but is impractical for large units. Wegelin (1988) reported that hydraulic filter cleaning only partially removed accumulated solids from horizontal flow roughing gravel filters in Peru.

On the other hand, upflow rock filters may be hydraulically cleaned and put into operation again if blockage occurs. In fact, water that has been filtered may be used to provide the shearing force necessary for backwashing. To provide the pressure for backwashing, an elevated tank may be utilized. Even the static head of water inside the filter may be employed for cleaning the filter. A fast

opening gate valve which will allow the water to flow downward is required to provide the hydraulic flush needed (Wegelin, 1988).

While Wegelin (1988) considered that horizontal flow roughing gravel filters possess an advantage of possible higher filtration rate (0.3-1.5 m/h) compared with vertical flow filtration (0.3-1 m/h), the reverse was stated by Schulz and Okun (1984). Also, Ives (1970) cited different works which proved that upflow rapid sand filters gave better results than downflow ones in treating wastewater effluents at rates of 8.5 and 12.5 m/h. Other work reported in the same reference, showed that both downflow and upflow sand filters were equally efficient at rates up to 12 m/h.

Upflow filtration has the advantages over downflow filtration of simplicity and low cost because the inflow and backwash structures are in the same direction and only one of them may serve the purpose of the other (Haney and Steimle, 1975). Also, these authors showed experimentally that filtration rates, for satisfactory performance of rapid sand filters, were higher in upflow filtration than in downflow.

2.2 Mechanisms of Filtration

Filtration includes physical, chemical and biological processes (ASCE et al., 1971). Biological growth may improve removal by reducing pore size but it is not precisely considered as a removal mechanism (Tchobanoglous and Eliassen, 1970). Camp (1964) stipulated that, for removal of suspended particles, it is necessary that contact takes place between these particles and the filter media or previously deposited material on that media. Two types of filtration mechanisms, namely transport and attachment mechanisms, have been identified by many authors (Ives,1970; 1975a; Cleasby, 1972; O'Melia, 1985). Table (2.1) shows a summary of the principal mechanisms that may be involved in removal of suspended solids during granular media filtration. Fig. (2.1) illustrates some of these mechanisms.

A combination of physical and biological factors are also thought to be responsible for algae removal in rock filters. The pores act as small settling

basins in which algae settle down to the rock surface or the slime layer formed by previous algal settling and subsequent endogenous respiration and bacterial degradation (O'Brien et al., 1973).

The mechanisms involved in removal of algae by rock filters are still not agreed upon among researchers and more investigation should be carried out in this regard (Middlebrooks, 1988).

Table 2.1: Filtration process variables and particle removal mechanisms.
Source: Tchobanoglous and Eliassen (1970).

Process variables	Removal mechanisms
1. Filter media grain size, shape, and density	Straining Mechanical Chance contact
2. Filter media porosity	Sedimentation
3. Media headloss, characteristics	Inertial impaction
4. Filter bed depth	Interception
5. Filtration rate	Chemical adsorption Bonding Chemical interaction
6. Allowable headloss	Physical adsorption Electrostatic forces Electrokinetic forces Van der Waals' forces
7. Effluent characteristics	Adhesion and adhesion forces
8. Chemical treatment	Coagulation-flocculation
9. Floc strength	Biological growth
10. Filter bed charge	
11. Fluid characteristics	

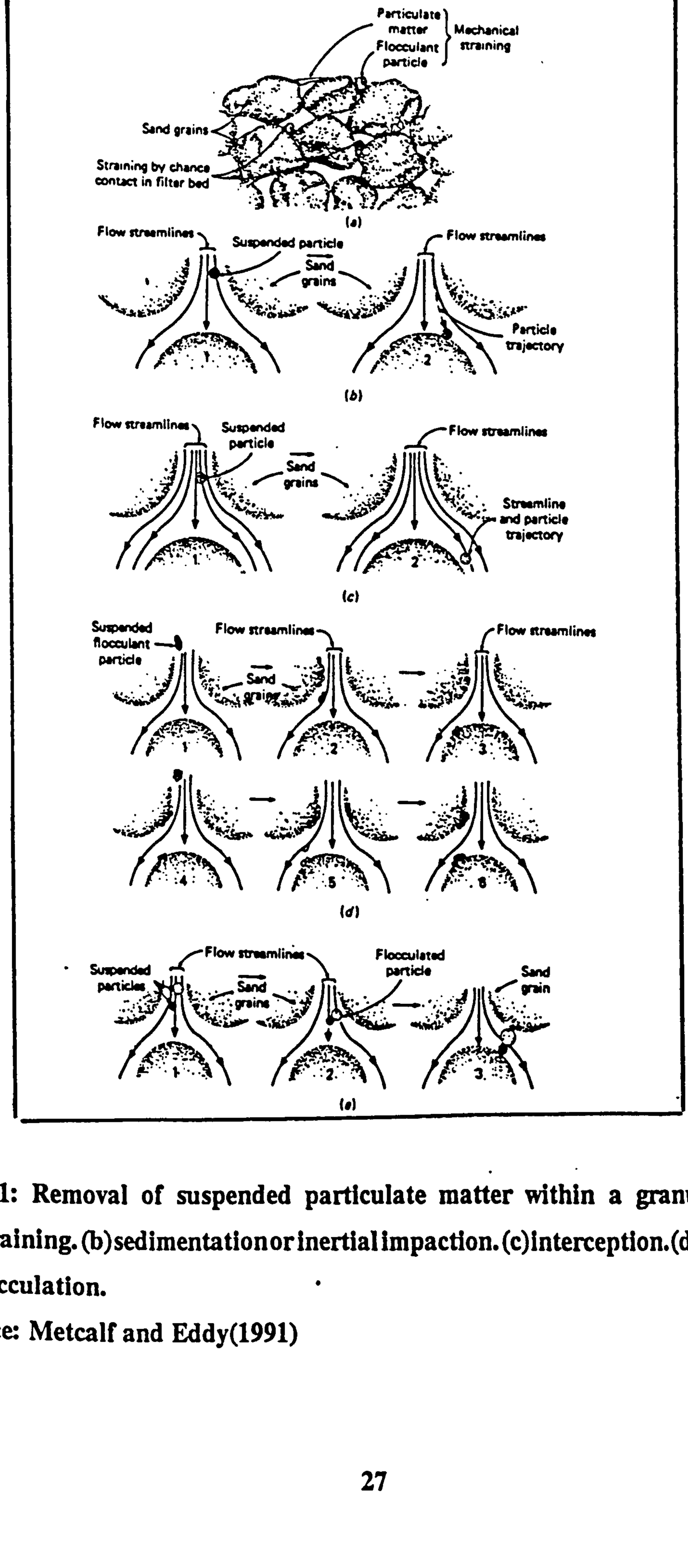


Fig.2.1: Removal of suspended particulate matter within a granular filter. (a)straining. (b)sedimentation or inertial impaction. (c)interception. (d)adhesion. (e)flocculation.

Source: Metcalf and Eddy(1991)

2.2.1 Transport Mechanisms

These mechanisms drive solids from the bulk fluid to contact with filter media or previously deposited material. They include gravitational settling, interception, hydrodynamic and inertial forces and diffusion. These are illustrated in Fig. (2.1).

(I) Sedimentation (Gravitational)

ASCE et al. (1971) stipulated that sedimentation is not so important as a mechanism of filtration. Also, the contribution of gravitational settling to the filtration process was found to be of minor importance by Jordan et al. (1974). Conversely, Van Vuuren (1981) described sedimentation as the most important mechanism in filtration, whether the flow is horizontal, downwards or upwards. Ison and Ives (1969) characterised sedimentation within a deep granular filter by the dimensionless group:

$$S = \frac{v_s}{v} = \frac{\textit{Stokes settling velocity of particle}}{\textit{approach velocity}} = \frac{g(\rho_s-\rho)d_p^2}{18\mu v} \quad ..(2.1)$$

Ives (1975a) calculated the ratio v_s/v_i, where v_i is the average interstitial velocity, for a 10 mm clay particle to be equal to 0.02, which would indicate that gravity has little effect in filtration especially at higher filtration rates. However, Ives elaborated further on the matter. He said that all components of the liquid velocity are proportional to approach velocity (v). One component of interest is the tangential velocity (v_t) which drives the particle away from the surface of the filter media. This component increases with the distance from the filter grains and diminishes to zero at the surface. So, if the ratio v_s/v_t is calculated for the same clay particle higher values will result especially if v_t is calculated near the surface of filter grain. This would indicate that settling is an important mechanism of filtration contrary to what may be interpreted from (v_s/v_i). Ives (1975a) concluded that the dimensionless group in Eq. 2.1 can be used to

describe the gravity mechanism but its numerical value should not be used to judge the collection efficiency.

In fact, experimental work by Ison and Ives (1969) showed that sedimentation is an important mechanism of removal in filtration. It was found that the removal coefficient was linearly related to the gravity parameter.

Discrete settling was originally assumed for algal cells in lakes and ponds (Bella, 1970). Later this assumption was extended to include rock filters where sedimentation was regarded as the basic mechanism of removal of algae in the pores of rock filters receiving the effluent of wastewater stabilization ponds (WWSP) at Veneta, Oregon (Williamson and Swanson, 1979; Stutz-McDonald and Williamson, 1979).

On the other hand, Cleave (1979) questioned this extension of discrete settling to algae in ponds. He argued that the species in ponds are different from those in lakes. Besides, 47% of the algae in the Veneta, Oregon ponds were motile and not readily settleable. In addition to that, Parker (1976) stated that removal by sedimentation is not probable for small algal species such as the green algae *Chlorella* and *Scenedesmus*, which are usually less than 20 microns.

The settling velocity of algae may range from 0.0 - 1.0 m/d. The lowest value is related to blue green algae, which become buoyant due to gas vacuoles produced by this type of algae under unfavourable conditions. Green algae and diatoms have better sinking rates (Bella,1970).

(II) Interception

Interception occurs when a particle follows a streamline that passes near the wall of the pore at a distance less than the particle's radius (Ives, 1975a; Metcalf and Eddy, 1979). Interception can be represented by the dimensionless group (Ison and Ives, 1969; Ives, 1970, 1975a):

$$I = \frac{d_p}{d_c} \quad \text{.....(2.2)}$$

where,

d_p= diameter of the particle

d_c= diameter of the collector (filter media)

The value of I was found to be between 2×10^{-4} and 1×10^{-1} (Ison and Ives,1969). When the value of I reaches one, straining becomes the operative mechanism. The mechanism of interception was shown to be of little importance in particle removal.

(III)Diffusion

Due to the thermal energy of water molecules, particles in water follow a random movement called Brownian motion. This motion may be opposed by increase of diameter over 1μm, where the drag ($3\pi\mu v_p d_p$, where v_p is the velocity of particle relative to fluid and d_p is its diameter) hinders the movement. Increase in temperature reduces viscous drag. An important characteristic of diffusion is the diffusion coefficient of the particle, B, which has the dimensions of velocity times distance. So, dividing by d_c, gives the mean velocity of particle caused by Brownian motion over a distance of one grain diameter. Dividing this Brownian velocity by the velocity of the particle caused by liquid, v (advective velocity) yields the reciprocal of the Peclet Number, P_e (Ives, 1970,1975a):

$$\frac{\text{Brownian velocity}}{\text{advective velocity}} = \frac{B}{d_c v} = \frac{1}{p_e} = \frac{\bar{K}T}{3\pi\mu d_p v d_c} \quad \text{....(2.3)}$$

B =the particle diffusivity from Einstein's equation for the diffusion coefficient of suspended particles, $\bar{K}T/3\pi\mu d_p$

d_c =collector diameter

v =approach velocity; particle velocity caused by liquid movement at an infinite distance from collector

P_e =Peclet number; ratio of transport by convective forces to transport by diffusion = vd_p/B

$\bar{K}$ =Boltzman's constant = 1.381×10^{-23} J/K

T =absolute temperature

μ =dynamic viscosity

d_p =diameter of the particle

The inverse of the Peclet number ($1/P_e$) ranges between $1x10^{-8}$ and 0.5×10^{-5} in water filtration (Yao et al.,1971). Diffusion is only effective as a removal mechanism when the suspended particles are less than one micron in diameter

(IV) Inertia

Inertia, also called impaction (Ison and Ives, 1969), refers to particles that continue moving by self kinetic energy and collide with filter grains following the original streamlines' direction before entering a filter pore. Streamlines converge when passing a filter pore. Stated differently, particles approaching a filter grain normally follow the flow that passes round it. However, particles possessing enough energy will continue moving and strike the grain surface, due to inertia. The ratio of the number of particles striking the grain to the number approaching it at an infinite distance upstream is given by the inertial efficiency dimensionless group, E (Ison and Ives, 1969; Ives, 1970; Ives, 1975a), where:

$$E = \frac{\rho_p d_p^2 U}{18\mu d_c} \quad ....(2.4)$$

ρ_p : density of particles, kg/m^3

d_p : diameter of particles, m

μ : dynamic viscosity, 1×10^{-3} $N.s/m^2$

d_c : diameter of collector (grain), m

U : velocity of fluid at infinite distance relative to the spherical grain, m/s. U was taken as the approach velocity, v, by Ison and Ives (1969).

The inertial efficiency group, E, was calculated by Ives (1975a) to vary between 2×10^{-9} and 1.5×10^{-3} . He concluded that inertia is negligible as a collection effect in filtration.

(V) Hydrodynamic Forces

If the flow in the filter pore is laminar with a velocity gradient, a shear field exists. Due to the shear gradient across the streamlines in a uniform shear field, a particle will have a differential velocity across its sides. This causes rotation and movement of the particle across the flow field. In a filter pore the shear regime is not uniform and particles are not spherical so, consequently, the particle is imbalanced further and the lateral forces are time dependant. Due to this difference in force, particles will move randomly across the streamlines and collide with filter grains (Ives, 1970, 1975a).

On the other hand, when particles approach a grain, the liquid trapped between them is displaced. Hence, this produces a viscous drag that tries to keep the particles away from the grain. However,this force does not affect the transport mechanism (Ives, 1975a). O'Melia (1985) called this force hydrodynamic retardation .

Ison and Ives (1969) characterised this mechanism by the Reynolds number $R_e = d_c V\rho/\mu$. They pointed out that other dimensionless groups similar to R_e could equally be used and R_e in terms of grain size alone was not satisfactory. However, they showed experimentally that hydrodynamic action expressed in terms of R_e was significantly effective in the removal of a kaolinite suspension by rapid sand filtration. A typical value of R_e in water filtration is 0.75. In fact, Ives (1975a) used $1/R_e$ to represent the hydrodynamic mechanism for random drift as produced by the dominance of viscous forces (μ):

$$\frac{1}{R_e} - \frac{\mu}{v d_c \rho_w} \quad \text{....(2.5)}$$

2.2.2 Attachment Mechanisms

These mechanisms cause particles to stick to the filtering media or to deposit layers of particles (Gregory, 1975). Also, they keep the attached particles from being swept again into the fluid moving within the pores. O'Melia (1985) stated that the drag force could reach infinity when the separating distance between the particle and the collector approached zero. So, other forces must exist for collision to occur. These are the attachment mechanisms. Gregory (1975) recognized that attachment mechanisms between particles and filter media are only required in the first stage of filtration. Of course, particles should continue to stick to deposited particles otherwise filtration will cease as the first coat is complete. The importance of attachment mechanisms and surface forces to removal in the filtration process was illustrated experimentally by Ghosh et al. (1975). These forces include adsorption, electrical interaction and chemical bridging.

(I) Adsorption

Adsorption is cited by the ASCE et al. (1971) as the most important mechanism. It occurs due to chemical and physical properties of both the media and the particles in the influent to be filtered. It depends on floc and filter grain size, adhesive property of the floc, chemical characteristics of the suspension and the shearing strength of floc. Contact between the floc and the grain occurs due to the convergence of streamlines at contractions in the pores. Adsorption could occur due to polymers existing in solution which bridge particles to filter grain (Ives,1970).

Foess and Borchardt (1969) found an electrophoretic mobility curve of algae similar to that of sand, where negative charge increased with increase in pH. This may suggest that algae will not be removed by sand filters because of

repulsion forces between the algae and sand grains. However, removal of algae by fine-grained (slow) and coarse-grained (rapid) sand filters has been shown to be feasible by many researchers, although rapid sand filters are inefficient (Ford and Tischler, 1976; Thanh and Pescod, 1976). Similar charges mean that electrostatic attraction could not occur. However, attachment of algae to sand grains, or to previously deposited algae, has been explained by Foess and Borchardt (1969) based on adsorption mechanisms which allow anionic polymers to adsorb onto negatively-charged surfaces. Mechanisms involved include one or more of ion interchange, hydrogen bonding, formation of coordinative bonding and linkages. Similar attachment mechanisms were explained by O'Melia and Stumm, (1967) during filtration when the surface charge on the suspended particles had the same sign as the surface of the filter media. Electrical double layer interaction and the resulting net attraction-repulsion has also been used to explain the attachment of like particles (Gregory, 1975). Although repulsive forces may prevail in water treatment, attractive van der Waals' forces may become significant due to compression of the double layer in filtration of sewage effluent (Ives,1970). This double layer model was also explained by O'Melia and Stumm (1967) and suggested as the basis of attachment when particles in the fluid being filtered are in the colloidal range.

(II) Electrostatic Attraction:

When the particles being filtered have surface charges opposite to those of the filter media, coulombic attraction is usually used to explain attachment.

2.2.3 Other Filtration Mechanisms

These mechanisms can neither be classified as transport mechanisms nor as attachment effects. They include straining and flocculation.

(I) Straining

Straining is removal on the face of filter media. It occurs mainly on precoat filters and in slow sand filters, where solids accumulate as a cake on the surface (Cleasby, 1972; Ives, 1975a). Interstitial straining also may occur in filters of coarse material where removal takes place in depth. It takes place when particles are filtered through pores whose opening is less than the diameter of the particles. However, rapid filters are generally not strainers except when particles are very large or the concentration of particles is high (Ives, 1970).

(II) Flocculation

This is not strictly a mechanism of removal. Flocculation within the filter will result in growth of particles which will enhance their removal by other mechanisms (Ives, 1970; Tchobanoglous and Eliassen, 1970). However, ASCE et al. (1971) stipulated that the importance of flocculation within the pores is of minor importance. Ives (1975a) using Smoluchowski's equation for particle aggregation and the Camp-Stein (1943) equation for mean velocity gradient, showed that the rate of particle aggregation within the pores is, with respect to depth, independent of flow rate and is directly proportional to pore surface/pore volume which is equal to $(6/d_c)^*(1-\epsilon)/\epsilon$, where d_c is the collector (filter grain) diameter and ϵ is the porosity of the media.

2.2.4 Detachment Mechanisms

Particles that are already deposited on filter collectors may find a way again into the bulk solution. The main reason for detachment is the shear force applied to deposited material due to an increase in interstitial velocity, v_i. This increase in velocity may occur due to a decrease in the size of the pores, because of deposition, or due to an increase in applied filtration rate. The shear strength of the deposit itself is also a fundamental factor in this detachment process (Ives, 1975a).

2.2.5 Combined Action of Mechanisms

The previously mentioned mechanisms do not act separately. For example, while particles are flocculating within the pores, interception, sedimentation, inertia and other mechanisms are working at the same time. O'Melia (1985) explained the relation between hydrodynamic retardation (drag on particles approaching collector walls) and other transport mechanisms. Hydrodynamic retardation is balanced by van der Waals' forces for small particles when diffusion is the mean transport mechanism. However, hydrodynamic retardation may reduce the deposition of larger, less dense, particles. Hence, for larger particles, where interception and gravity are effective transport mechanisms, hydrodynamic retardation is more pronounced and neglecting it will result in a discrepancy between theory and experimental results.

Ison and Ives (1969) found that removal efficiency is proportional to $(d_p/d_c)^{-2.3}$; the group which represents removal by interception. This indicates that a higher (d_p/d_c) ratio results in lower efficiency. Large particles are subjected to greater shearing action and more drag as they approach the wall of the collector. Also, they have less diffusion behaviour. However, efficiency will not improve or decline infinitely as (d_p/d_c) decreases or increases successively. These are upper and lower limits where the mechanisms of filtration change from diffusion at lower (d_p/d_c) ratios to straining, when the ratio becomes equal to one (Ison and Ives, 1969; Ives, 1970, 1975a). This agrees with the findings of O'Melia and Stumm (1967) and Yao et al. (1971). Because different mechanisms operate together and some mechanisms become dominant, depending on factors such as diameter of the particle, viscosity, diameter of the collector, they concluded that a particle diameter which corresponds to minimum efficiency of removal must exist. They proved experimentally that this particle diameter is 1μm.

Ives (1975a) summed up transport mechanisms into an expression of filter efficiency. He defined a removal coefficient Λ which relates to λ, the filter coefficient first introduced by Iwasaki (1937) in his model (Eq. 2.6) for filtration in depth:

$$\frac{\partial C}{\partial x} = -\lambda C \quad .....(2.6)$$

$$\Lambda = \lambda d_c \quad .....(2.7)$$

$$\Lambda \propto (Interception)(diffusion)(settling)(hydrodynamics)..(2.8)$$

C = concentration of suspended solids at any time at any depth in the filter.

x = length of filter.

λ = filter coefficient.

d_c = diameter of the collector (filter grain)

Rewriting Eq. 2.6 and 2.7 using $\Delta C = \partial C$, $\Delta x = \partial x$ and considering that C represents the concentration flowing into layer Δx :

$$\Lambda = const. \left(\frac{-\Delta C}{\Delta x}.d_c\right)/C \quad ...(2.9)$$

In Eq. 2.9, $\Delta C/\Delta x$ is the concentration change per unit filter length and $(\Delta c.d_c)/\Delta x$ is the concentration change in a layer whose thickness is the diameter of the filter media. Hence the removal coefficient, Λ, is the fraction of solids retained in a filter layer one grain diameter thick.

Inserting into Eq. 2.8, Eq. 2.1, 2.2, 2.3 and 2.4 , which represent settling, interception, diffusion and hydrodynamics, respectively, results in Eq. 2.10:

$$\Lambda = const. \left(\frac{d_p}{d_c}\right)^{\alpha} \left(\frac{\bar{k}T}{3\mu d_p v d_c}\right)^{\beta} \left(\frac{g(\rho_s - \rho_w)}{18\mu v}\right)^{\gamma} \left(\frac{\mu}{v d_c \rho_w}\right)^{\delta} \quad ... (2.10)$$

$$\Lambda = const. \frac{d_p^{\alpha-\beta+2\gamma}}{(\mu^{\beta+\gamma-\delta})(d_c^{\alpha+\beta+\delta})(v^{\beta+\gamma+\delta})}.(\bar{k}T)^{\beta}.\frac{(\rho_s - \rho_w)^{\gamma}}{\rho^{\delta}} \quad ...(2.11)$$

Eq. 2.11 may be more helpful to interpret the combined action of different mechanisms whose relative importance is dependant on particle properties such as shape and density and on fluid flow conditions in addition to the geometry of the filter pores. In general, smaller approach velocity and smaller filter grains give better results.

The diameter of the particle appears in both the numerator and denominator of Eq. 2.10. This indicates that efficiency increases with increase in d_p when d_p is large and density is high such that settling is the effective transport mechanism. When d_p is very small, diffusion becomes important,i.e. β is large. When the case is such, decrease in d_p will improve efficiency. This again backs up the findings of Yao et al. (1971). They concluded that a particle diameter which will be removed at minimum efficiency, must exist. A particle of 1 μm diameter was found to match with minimum efficiency.

If $\beta + \gamma > \delta$, then an increase in temperature will improve removal provided that either particles are large enough to settle effectively, or small enough to diffuse efficiently, and the approach velocity is relatively small. When this condition occurs, increase in temperature will decrease viscosity and, consequently, removal will be enhanced because diffusion and settling will overcome hydrodynamic forces.

Ison and Ives (1969) showed experimentally that the value of the approach velocity has the most influence on filter performance. They found that λ could be written as:

$$\lambda = \frac{\mu^{1.4} d_p^{0.3}}{d_c^{1.4} v^4} \quad \ldots \quad (2.12)$$

2.3 Factors Affecting Algae Removal by Rock Filters

Middlebrooks (1988) concluded that the impact of design parameters on rock filter efficiency is still uncertain which explains the inconsistencies experienced with the performance of rock filters in different locations. He

recommended further research in this field. Table (2.2) summarizes the design and performance of some rock filters that are reported in the literature.

2.3.1 Chemistry Of Particle And Solution

The chemical composition of a particle affects its attachment to the grains of the filter. Cellulose constitutes 46% of *Chlorella* cells while protein forms 27%. This structure causes the repulsive force of *Chlorella* to decrease in the acidic pH range due to gaining of H^+ ions by NH_2 in the protein. Increase in repulsion will occur due to increase of surface negative charge on algal cells which results in a decrease in the removal efficiency of algal cells in sand filtration (Foess and Borchardt, 1969).

2.3.2 Rock Size

Regarding sand filters, it is established that the process of filtration is affected by the size of the media (Fair et al., Jordan et al., 1976; 1968; McGhee, 1991). It was also mentioned in Table (2.1) that grain size is one of the variables that affect the filtration process.

The effect of rock size in relation to other factors (such as hydraulic loading) which affect removal of algae is not well defined in the literature. Nevertheless, O'Brien (1975, 1976) suggested a design hydraulic loading of 1203 $l/m^3.d$ for rock size between 2.54 cm and 12.7 cm. He recommended that this figure should be decreased if a larger rock was used.

Table (2.2) shows that the size of rock used as a filling material in the design and construction of rock filters covers a wide range between 0.47 and 20 cm. There is no reported case in the literature reviewed where unisize rock was employed as the filtering medium for algae removal by the rock filtration process. The absence of this type of construction did not allow workers in this field to define the relation between rock size and rock filter efficiency.

Table 2.2: Summary of design and performance characteristics of some rock filters treating wastewater stabilization pond effluent at different locations

REFERENCE	LOCATION	HYDRAULIC LOAD m3/m3/d	ROCK SIZE cm	NOTES
Swanson and Williamson (1980) Williamson and Swanson (1979)	Veneta Oregon	0.06 , 0.07 , 0.13 , 0.17 , 0.27 , 0.28 ,	7.6 - 15.2	-horizontal flow -field scale -depth of rock = 1.8 m . -water depth = 1.5 m . -effective surface area = 5400 m2 . -effective volume = 8200 m3 . -therotical detention time = 1.6 days. -ammonia increased during warmer months. -porosity = 0.42
	Veneta, Oregon	ranges between 0.12 - 0.53	7.6 - 15.2	-horizontal flexiglass channel . -pilot scale -effective height = 0.38 m . -length = 7.6 m -width = 0.45 m
Middlebrooks (1988)	Eudora Kansas	1.2 in Summer 0.4 in Winter	1.3 in the first filter 2.5 in the second filter	-horizontal flow -field scale -submerged rock depth = 1.5 m . -effluent from the bottom. -peak efficiency in Summer and early Fall. -TSSin = 40-70 mg/l . -BOD5in = 10-35 mg/l . -the filters met the criteria of 30 mg/l - BOD5 but did not consistantly meet 30 mg/l of TSS.
	California, Mo.	design = 0.4 actual } average} = 0.25		-horizontal flow -field scale -TSSin = 69 mg/l . -TSSout average = 22 mg/l . However,it failed to meet 30mg/l-TSS in 11 samples out of 19.
	Veneta Oregon	0.07 , 0.13 0.14 , 0.18 0.27 , 0.28	7.5 - 20.0	-horizontal flow -field scale -depth = 2. m . -volume =757 m3 . -met daily max. BOD5 of 20mg/l and TSS of 20 mg/l for max. hydraulic loading of 0.3 m3/m3.d . -hydraulic loading affected performance .
	Illinois	0.8	8 - 15	-horizontal flow -field scale -all figures are design criteria based on the operation of 13 rock filters . -rock is 0.3 m above maximum water level . -post aeration ability is necessary . -hudraulic loading < 0.8 m3/m3.d did not seem to improve the performance . -difficulties were met with rock diameter less than 7.6-15 cm and with flat rocks .
	West Monroe, La.	0.36	5 - 13	-horizontal flow -field scale -filter depth = 1.8 m . -dischrge is intermittent . -periodically,effluent concentrations exceeed 30mg/l for TSS and BOD5 but not simultaneously . -performance does not seem to be related to hydraulic loading .
O'Brien (1976) I)LARGE ROCK	Eudora, Kansas	0.5 , 0.85 1.0 , 1.2 1.5 , 1.8 2.1 , 2.9	0.95 - 5.08	-horizontal flow . -field scale -rock depth = 1.4 m . -water depth = 1.22 m . -three grab samples/week -TSSout > 30 mg/l consistently . -NH3 in effluent increases in winter . -greatest TSS removal and anaerobic conditions occur in Summer and early Fall. -it is recommended to use rock size between 2.54-12.7 cm with majority around 5 cm . -max. recmmended hydraulic loading=1.2 m3/m3.d . This figure should be reduced if rock size is increased. -porosity=0.44
II)SMALL ROCK		0.062 , 0.4 0.5 - 1.1 1.4 - 2.2	0.47 - 2.54	
O'Brien et al. (1973)			2.5 - 5	-upflow -lab scale -ammmonia decreased in effluent -nitrate and nitrite increased in effluent -37% removal of total COD -27% removal of soluble COD

2.3.3 Influent Suspended Solids Concentration.

Data presented by Williamson and Swanson (1979) show that effluent TSS kept below 14 mg/l irrespective of the influent TSS, which ranged between 22 and 105 mg/l.

Influent total suspended solids levels considered in the work of O'Brien (1975,1976) were approximately 60 mg/l. This did not allow any conclusion on whether there is a relationship or not between removal efficiency and TSS concentration in the influent to rock filters.

The highest reported TSS applied to rock filters was 78 mg/l, a fact which did not help to evaluate the effect of this factor on rock filters' performance (Parker, 1976).

2.3.4 Hydraulic Loading

O'Brien (1975,1976) proved that lower hydraulic loading increased the efficiency of TSS removal. To offset the negative effect of winter on the performance of the rock filter, he recommended a design hydraulic loading of 400 $l/m^3.d$ compared with 1203 $l/m^3.d$ in Summer.

Observations made by Middlebrooks (1988) show discrepancies regarding the effect of hydraulic loading on rock filter performance. He mentioned that the performance of rock filters at Veneta, Oregon was affected by hydraulic loading. However, the hydraulic loading applied was very low (0.07-0.28 $m^3/m^3.d$). On the other hand, loading less than 0.8 $m^3/m^3.d$ did not seem to improve the quality of the effluent in Illinois. Also, it was reported that performance did not show any relationship with hydraulic loading at West Monro, L.A. (ibid).

Williamson and Swanson (1979) found a linear correlation between TSS percentage removal and hydraulic loading. Removal efficiency ranged between 50% and 100% with hydraulic loading 0.28-0.07 $m^3/m^3.d$. However, this work and all the previously cited work did not relate hydraulic loading mathematically to other factors which may affect the process of rock filtration. They did not establish mathematically the relationship between hydraulic loading and rock size

for example. Besides, the linear correlation found by Williamson and Swanson (1979) is restricted to a low range of loading rate.

2.3.5 Algal Species

Different algal species have different shapes, sizes and motility properties. This produces a variation in the degree to which each algal species is removed. Ramani (1976) stated that filamentous algae (e.g *Oscillatoria*) may be removed more easily than nonfilamentous types (*Chlorella, Scenedesmus*). He added that diatoms are the most difficult to remove from stabilization pond effluent.

Most species of algae are denser than water and settle under quiescent conditions, with a terminal settling velocity called the sinking rate. The sinking rate depends on the size, density, shape and species of algae (Titman, 1975). The shape of algal cell is related to the amount of form drag on the cell during its sinking (Boney, 1989). However, the effect of shape on settling velocity is size-dependent. For particles of 5μm diameter, the settling rate is arranged in decreasing order as: plate > cylinder > sphere; for 50 μm: cylinder ≈ plate > sphere (Smayda, 1970).

Different types of algae have different settling velocities and, consequently, different removal efficiencies. The sinking rates of algae range between zero and one m/d. The lowest rate represents blue green algae, which becomes buoyant due to gas vacuoles produced by this type of algae under unfavourable conditions. Green algae and diatoms have better sinking rates (Bella, 1970). This does not agree with the above mentioned suggestion by Ramani (1976) that diatoms are the most difficult to remove. Also, Palmer (1980) pointed out that diatoms will settle under quiescent conditions.

Algal cells differ also in size. In general, larger cells have faster sinking rates and are more easily removed (Eppley et al., 1967; Smayda, 1970; Titman and Kilham, 1976).

The movement of some algae was reported to help them escape capture in filter pores. Parker (1976) suggested that the gliding movement of *Oscillatoria* due to contractions in the cells and the flexible deformable cell wall of *Euglena*

may allow these types of algae to pass around obstacles. Smayda (1970) and Cleave (1979) also indicated that motile algae do not settle easily.

In contrast to these findings regarding the effect of algal species on algal removal, Williamson and Swanson (1979) observed that neither settling rate nor removal of algae by rock filters at Veneta, Oregon changed with varying species.

2.3.6 Temperature

It was suggested by O'Brien (1975) that under cold winter conditions, biological activity in the rock filter is minimized and the removal of algae is confined to sedimentation.

Smayda (1974) and Stutz-McDonald and Williamson (1979) found that the sinking rates of algae increase hyperbolically with temperature. However, the factor of increase in sinking rate (2.5-7.5) could not be explained by the effect of temperature on viscosity, which decreased by a factor of only 1.5 when temperature was raised. The latter authors suggested that lower temperature changed the physiological characteristics of algal cells such that removal decreases at lower temperature more markedly beyond what can be explained by viscosity alone.

On the other hand, Williamson and Swanson (1979) found that the settling rate of algae did not vary with temperature or other environmental conditions. They suggested that decrease of sinking rate in cold weather may have been made up for by the presence of new algal species with faster settling rates.

2.3.7 Other Factors

These include stresses such as nutrient deficiency, age, growth phase, whether the algae are alive or dead, light intensity, and aerobic or anaerobic conditions.

Non-motile algae, flagellated algae and diatoms will sink overnight (Smayda, 1970). On the other hand, cells of blue green algae form gas vacuoles

in a light dependent process. In dim light gas vacuoles increase causing these cells to float. In bright light photosynthesis is increased which leads to increased pressure in the cytoplasm surrounding these gas vacuoles with the result that these cells sink (Boney, 1989). In general, some inhibition of photosynthesis occurs at higher light intensities; maximum photosynthesis occurs at medium light intensity (0.03-0.06 cal/cm^2.min). In darkness, photosynthesis will stop after a certain period (Jackson, 1967).

Titman and Kilham (1976) found that the settling rate of certain species of algae is not constant. Cells which suffer nutrient depletion sink at higher rates, 2-to-4 times more, than those in nutrient abundance conditions. Nutrient-limited conditions during the stationary phase of growth lead to higher settling rates relative to the exponential growth phase. Similar findings were reported by other workers (Eppley et al., 1967; Smayda, 1970; 1974). Also, dead and old algal cells were found to sink faster (Smayda, 1974).

Williamson and Swanson (1979) assumed that there is no difference in settling rate under aerobic or anaerobic conditions. Cleave (1979) did not agree with this assumption. However, the experiments carried out by Stutz-McDonald and Williamson (1979) revealed that there is no significant difference in the settling rate of algae under aerobic or anaerobic conditions.

2.4 Deep Bed Filtration Mathematical Models

There are two fields usually investigated in relation to flow of liquid into a porous media. The first is concerned with the hydraulics of movement of fluids through the porous media. This branch has been investigated by many researchers (Fair and Hatch, 1933; Carman, 1937, 1939; Rose, 1945, Rose and Risk, 1949; Ergun, 1952; Ranz, 1952; Neale and Nader, 1974; Spielman, 1975). The second branch applies to liquids containing suspended material that is to be removed by passage through porous media; this process is known as filtration.

In attempting to describe the process of filtration by mathematical models, two directions can be distinguished (Cleasby, 1972; Amirtharajah and Wetstein, 1980): the macroscopic approach and the microscopic approach. The

first approach did not emphasize the mechanisms of the filtration process. Instead, it dealt with relating macroscopic variables, such as grain size, rate of filtration and water viscosity, to predict the time-space variation of water quality and head loss through the filter media. The macroscopic technique is also known as the physical filtration theory (O'Melia and Stumm, 1967). The second method is concerned with the mechanisms of removal and it has also been termed the chemical filtration theory because it concentrates on the chemistry of the particle to be removed and the surface chemistry of the filter material (ibid).

2.4.1 The Basic Equations of Iwasaki

Originally, studies concerned with turbidity removal dealt with the filter as a whole unit without attention being given to depth. Of the first studies to consider variation of removal with respect to depth and variation of removal at any depth with respect to time was one by Iwasaki (1937) and another by Eliassen (1941).

Eliassen (1941) concluded that removal occurs throughout the depth of filter and, although the first layer of the filter performs most of the removal at the beginning of a filter run, removal goes towards zero at the end of the filter run and a higher contribution is gradually made by the rest of the filter .

Earlier, Iwasaki (1937) presented the fundamental principle upon which many researchers based their models later on. He stated that the rate of removal of suspended solids per unit depth of filter is proportional to the concentration of particles present at that location or entering that unit depth. This principle was previously shown mathematically in Eq. 2.6 and repeated here for convenience:

$$\frac{\partial C}{\partial x} = -\lambda C \quad \text{.....(2.6)}$$

Where, λ is the filter coefficient or the impediment modulus, the original term used by Iwasaki. Other terms are as previously defined.

Iwasaki (1937) also introduced two other equations. The first is the continuity equation (2.13); the second is to define λ (Eq. 2.14):

$$v\frac{\partial C}{\partial x}+\frac{\partial \sigma}{\partial t} = 0.0 \quad .....(2.13)$$

$$\lambda = \lambda_{\circ}+C_1\sigma \quad .....(2.14)$$

C = concentration of TSS at any time at any depth in the filter, mg/l.

v = superficial (approach) velocity, l/min.cm^2).

σ = specific surface deposit; material deposited per cm^3 of the filter media, mg/cm^3.

$\lambda_{\circ}$= initial filter coefficient,cm^{-1}.

x = filter depth,cm.

C_1= constant.

Eq. 2.13 states that the amount of material accumulated in the filter per unit volume of filter media per unit time is equal to the amount of removal from suspension per unit volume of liquid per unit time.

Hall (1957) considered λ as a constant. Other researchers found that it varies with time and depth (Camp, 1974; Ives, 1969, 1975b; Tchobanoglous and Eliassen, 1970).

Eq. 2.14 shows that the filter coefficient, λ, increases as specific deposit increases.

2.4.2 Ives' Equations

Ives has published a notable amount of work on filtration theory (see for example Ives,1961, 1964, 1966, 1975 and 1986).

Ives utilized the basic equation of Iwasaki (Eq. 2.6) to describe the variation of concentration with respect to depth of filter. For finding an expression of λ, Ives followed a rational approach based on two opposing factors

affecting the value of λ: the specific surface (surface area per unit filter volume) and the interstitial velocity.

Ives assumed that the effect of specific surface is a combination of the effects of spherical pores and cylindrical pores. At the beginning, λ increases with deposition, which increases the specific surface of the spheres. However, when the pores become cylindrical due to deposit accumulation, further deposition makes the capillaries smaller and λ decreases. This was expressed mathematically by Eq. 2.15:

$$\lambda = \lambda_{\bullet}\left[1+\frac{\beta_p \Omega}{\varepsilon}\right]^{2/3}\left[1-\frac{\Omega}{\varepsilon}\right]^{1/2} \quad .....(2.15)$$

λ = filter coefficient (length $^{-1}$)

$\lambda_{\bullet}$= filter coefficient for the clean filter when Ω=0.0.

ε = porosity of the clean filter (dimensionless).

β_p= packing constant = $\varepsilon/(1-\varepsilon)$= void ratio (dimensionless).

Ω = volume of specific deposit per unit filter volume (dimensionless).

Eq. 2.15 deals only with the effect of specific surface on the filtration process. The second part of the model is the effect of interstitial velocity. Ives suggested that the specific surface would not continue to decrease to zero and all the voids become filled with deposits, $\Omega=\varepsilon$. He assumed a critical velocity is reached, v_c, when an ultimate specific deposit, Ω_u, is attained:

$$v_c = \frac{v}{\varepsilon - \Omega_u} \quad .....(2.16)$$

Also, since clarification is inversely proportional to approach velocity, Ives assumed that:

$$\lambda \propto \left[\frac{1}{v_i}-\frac{1}{v_c}\right]^{j} \quad .....(2.17)$$

From Eq. 2.16 and 2.17 the velocity term becomes:

$$\lambda = \lambda_o \left[1 - \frac{\Omega}{\Omega_u}\right]^j \quad \text{.....(2.18)}$$

Since the pores of a filter are not such the ideal shapes of spheres or cylinders, Ives suggested that the powers in Eq. 2.15 be replaced by constants, f and i, to make the model more general.

Combining the surface terms with the velocity term into Iwasaki's equation (2.6), with C_v representing the volume/volume concentration of suspended matter:

$$\frac{\partial C_v}{\partial x} = -\lambda_o \left[1 + \frac{\beta_p \Omega}{e}\right]^f \left[1 - \frac{\Omega}{e}\right]^i \left[1 - \frac{\Omega}{\Omega_u}\right]^j \quad \text{.....(2.19)}$$

2.4.3 Eliassen's Equations

Eliassen (1941) proved experimentally that removal is not restricted to the upper part of a downflow filter but all the filter unit takes part in the process of removal of suspended matter from water.

Tchobanoglous and Eliassen (1970) concluded that the rate of change of concentration with distance in the filter is proportional to a coefficient called the retardation factor. This factor is changing with distance and the removal achieved in the filter. They assumed that the probability of removal within the first layer is higher than the second because in the first layer a larger particle range is passed. This was mathematically expressed in Eq. 2.20 utilizing Iwasaki's equation (2.16).

$$\frac{dc}{dx} = -\left[\frac{1}{(1+ax)^n}\right] r_o C \quad \text{.....(2.20)}$$

The term between brackets is the retardation factor, r_o is the initial removal rate (length^{-1}), a and n are constants. Other terms are as previously defined.

It was observed that r_o is related to the size and distribution of the particles. Also, the value of n was related to particle size in the influent. For uniform filter grains and particle size, n is equal to zero and the initial removal is described by a first order equation. When the value of n becomes equal to 1, the retardation coefficient decreases steeply in the first layers of the filter, then more gradually in the rest.

To take into consideration the effect of deposited material on removal efficiency, another term was added into Eq. 2.20 which resulted in Eq. 2.21:

$$\frac{dc}{dx} = -\left[\frac{1}{(1+ax)^n}\right] r_o C \left[1-\frac{q}{q_u}\right]^m \quad \text{.....(2.21)}$$

In Eq. 2.21, q represents the mass of suspended solids deposited in the filter, q_u is the ultimate mass that can be deposited in the filter before breakthrough occurs and m is a constant related to strength of the suspended matter.

Eq. 2.21 shows that removal always decreases as the deposition builds up. It does not consider the improvement in removal efficiency due to increase of specific surface area, as was shown by Ives's equations. When $q=q_u$ the rate of change of concentration with respect to depth becomes equal to zero.

Eq. 2.20 and 2.21 were derived for filters where straining is the operating mechanism. However, Tchobanoglous and Eliassen (1970) concluded that these equations may also be applicable when other mechanisms are responsible for removal.

2.4.4 Microscopic Filtration Theory

This theory concentrates on the chemical properties of the solution and the surface of the filter bed and deals with the basic mechanisms of removal; thus it is also called the chemical, and sometimes the conceptual model, of filtration.

The model was developed by O'Melia and Stumm (1967), Yao et al. (1971), O'Melia and Ali (1978), also O'Melia (1985).

The microscopic model was based on the assumption that removal of particles occurs due to collision of particles with the filter media in a manner which resembles what is occurring in the flocculation process. Filter grains serve as collectors on which particles are captured. This requires that particles be transported towards these collectors. An attachment mechanism is also required to make particles adhere to these collectors. These mechanisms have been previously explained in this Chapter.

In order to describe the efficiency of each mechanism in particle removal, a single collector efficiency factor, η_c, was defined for each mechanism. This factor, η_c, may be defined as the rate at which particles strike the collector divided by the rate at which particles flow towards the collector. Originally, the correlation between filtration mechanisms and single fibre efficiency was accomplished by Friedlander (1958) for aerosol filtration by fibrous filters. It was extrapolated for sand filtration by O'Melia and Stumm (1967).

For Brownian diffusion, interception and gravity sedimentation, the single collector efficiency factors $(\eta_c)_D$, $(\eta_c)_I$, and $(\eta_c)_G$ were defined, respectively, as follows in Eq. 2.22, 2.23 and 2.24. All terms appearing in these equations are as previously defined.

$$(\eta_c)_D = 4.04P_e^{-2/3} = 0.9\left(\frac{\bar{k}T}{\mu d_p d_c v}\right)^{2/3} \quad \text{.....(2.22)}$$

$$(\eta_c)_I = \frac{3}{2}\left(\frac{d_p}{d_c}\right)^2 \quad .....(2.23)$$

$$(\eta_c)_G = \frac{g(\rho_s - \rho_w)d_p^2}{18\mu v} \quad .....(2.24)$$

The overall single collector efficiency factor (η_c) is the sum of these factors:

$$\eta_c = (\eta_c)_D + (\eta_c)_I + (\eta_c)_G \quad .....(2.25)$$

To describe the change of concentration with respect to depth, Iwasaki's equation (2.6) was used and the result is expressed in Eq. 2.26:

$$\lambda = \frac{3}{2}\left[\frac{(1-e)}{d_c}\right]\alpha\eta_c \quad .....(2.26)$$

Where, α is the collision efficiency factor; it is defined as the ratio of collisions that result in particles sticking to the collector divided by the number of collisions which occur between suspended and the filter media. It reflects the chemistry of the system and is usually taken as equal to 1.

Taking the influent concentration as C_{in}, then the concentration at any depth at any time C is represented by Eq. 2.27, which is a solution of Eq. 2.6 and 2.26:

$$C = C_{in}\exp\left[\frac{-3}{2}(1-e)\alpha\eta_c\left(\frac{x}{d_c}\right)\right] \quad ....(2.27)$$

2.4.5 Williamson and Swanson Model of the Horizontal Flow Rock Filter.

Williamson and Swanson (1979), also Swanson and Williamson (1980) assumed that algae are removed in horizontal flow rock filters by discrete settling. They concluded that the percentage of removal of total suspended solids is a function of hydraulic loading. An inverse linear relationship was found between hydraulic loading and percentage solids removal for the hydraulic loading range of 0.07 to 0.28 $m^3/m^3.d$.

A model was developed based on the introduction of the critical settling velocity v_c, which is the velocity of a particle that will settle a distance d_s during detention time, θ (Fig.2.2):

$$v_c = \frac{d_s}{\theta} \quad \text{.....(2.29)}$$

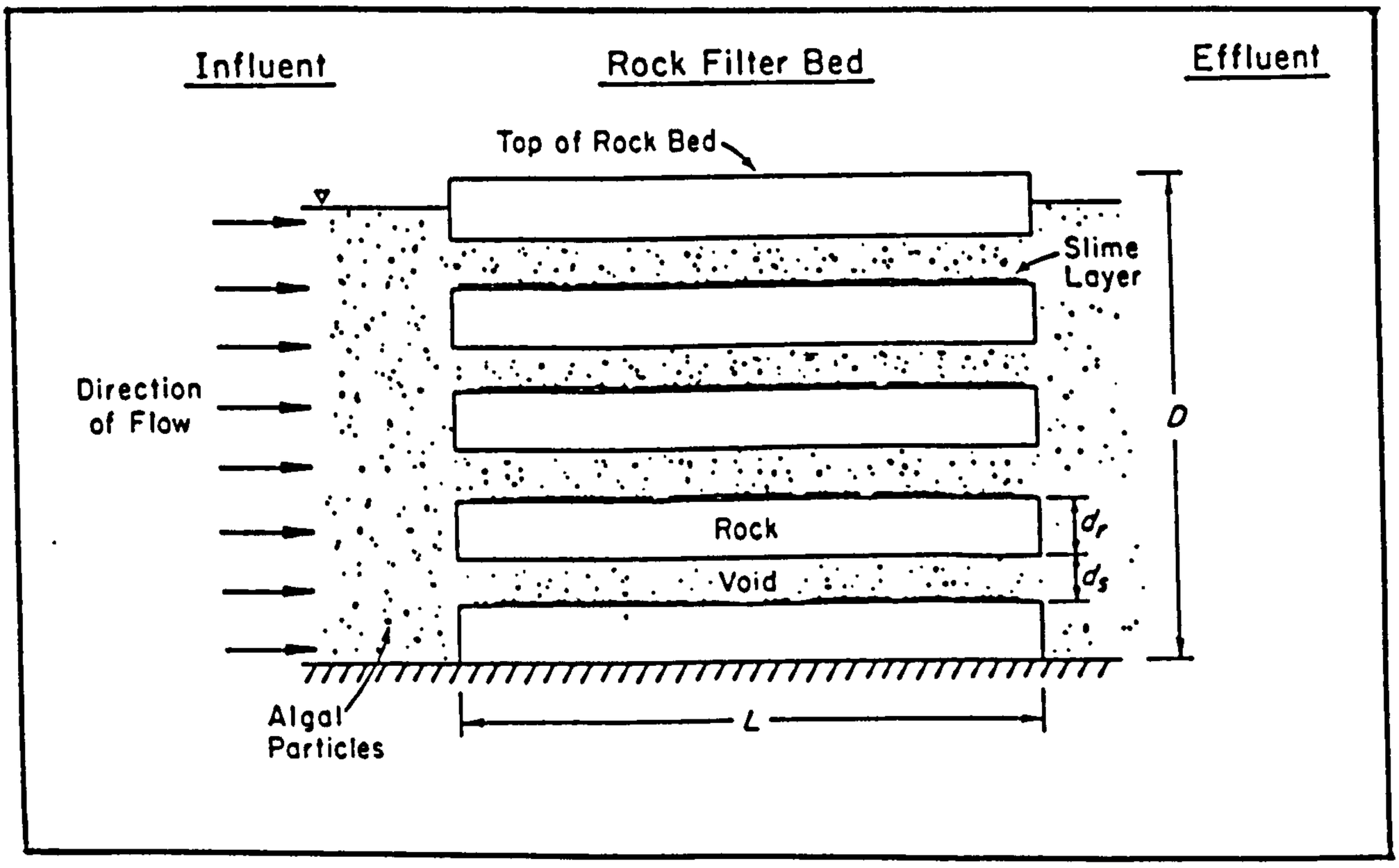


Fig.2.2: Horizontal flow rock filter conceptual model.

Source: Swanson and Williamson (1980)

Eq. 2.30 was formulated using the principles of sedimentation theory (Camp, 1946), that particles of settling velocity v_p greater than v_c will be completely removed and those with $v_p < v_c$ will be removed in the ratio (v_p/v_c). This yields:

$$R = P_c + \frac{\theta}{d_s} * \sum_{v_p=0}^{v_c} v_p P_p \quad \text{.....(2.30)}$$

R = removal efficiency.

P_c = fraction of algal particles completely removed.

P_p = fraction of algal particles with settling velocity v_p.

To apply this model it is necessary to determine the velocity distribution of algal particles present. Also the settling distance, d_s, has to be determined.

2.5 Summary

Based on the previous review of literature, a study of rock filters for algae removal must be designed such that mechanisms of removal may be recognized. Also, experiments should be performed in a manner that helps to identify how performance of the filters is affected by different variables, such as rock size, filter depth and hydraulic loading.

Upflow rock filters have received little attention by researchers and it is reasonable to study this mode of operation, especially since this configuration of filters has the advantage that it allows practical backwashing when blockage occurs.

CHAPTER 3

Removal of Algae by Coagulation

The terms coagulation and flocculation are used in scientific literature with different definitions. Generally, they involve aggregation of colloidal particles into larger particles or flocs.

Colloids or colloidal systems are composed of a homogeneous medium and colloidal particles dispersed there-in (Jirgensons and Straumanis, 1964). The definition of colloidal particles is based on their size, being those particles in the size range 10^{-3} to 1μm (Shaw, 1980; Barnes et al., 1981; and Silcocks, 1982) although Glasstone and Lewis (1970) considered them to lie between $5x10^{-3}\mu$m and 0.2 μm. An even higher size range of 1μm-500μm was assigned to colloidal particles by Jirgensons and Straumanis (1964). However, it should be noted that these limits are approximate and there is no sharp distinction between colloidal and non-colloidal systems; from a molecular solution to a colloidal dispersion, at one extreme, and from a colloidal dispersion to a suspension at the other extreme (Glasstone and Lewis, 1970; Shaw, 1980; Silcocks, 1982; Tan,1982).

The aggregation of colloidal particles involves two steps: transport, in order that contact occurs, and destabilization, which causes attachment. Coagulation comprises both processes of aggregation. Flocculation refers specifically to the transport step (O'Melia, 1972, 1978, 1978a; Gregory, 1979; Barnes et al., 1981; AWWA, 1982; Lawler, 1986). In this sense these terms will be used here although other definitions have been given by different authors.

Some writers used the two terms synonymously to mean agglomeration of colloidal particles by addition of electrolytes (Camp, 1955; Van Olphen and Mysels, 1975). Stevenson (1980) described chemical destabilization of dispersions and formation of initial nuclei as coagulation. Flocculation was defined as the set of physical processes whereby destabilized material is agglomerated to form separable flocs. Similar definitions were given by Steel and McGhee (1979), Bratby (1980), Alaerts and Van Haute (1982) and Reynolds (1982).

La Mer (1964) and Lyklema (1978) based their definition on mechanisms of destabilization and coagulant used. Destabilization by double layer compression, which is usually caused by ions or inorganic hydrolysed species,was referred to as coagulation. Flocculation was assigned to destabilization and aggregation by formation of bridges between particles and the polymers added. Tekippe and Ham (1971) defined the terms in a similar way.

If transport is caused by Brownian motion of the particles, then it is referred to the aggregation process as perikinetic flocculation.When fluid motion and velocity gradient are responsible for transport of particles, the process is known as orthokinetic flocculation (Tekippe and Ham, 1971; Amirtharajah, 1978).

3.1 Colloidal Systems

Some authors divide colloidal systems according to the relation between the solvent and solute, into lyophobic, solvent-fearing, and lyophilic, solvent loving (Sheeham,1970; Van Olphen and Mysels, 1975). When the solvent is water these become hydrophobic and hydrophilic colloidal systems (O'Melia, 1972). Sols, emulsions, aerosols and air-borne dust are examples of the first category. Lyophilic colloids are either solutions of macromolecules, such as polymers and electrolytes, proteins and polysaccharides, gels, glue, and biological systems.

Shaw (1980) classified colloids into three systems:

(1) Colloidal Dispersions: These are thermodynamically unstable (irreversible) systems. They are further classified as shown in Table (3.1) according to the phase of the matter which composes the "disperse" (the solute or dispersed phase, which is the material that is dispersed in the form of particles) and the "disperse medium" (solvent; the material in which particles are dispersed).

(2) True Solutions of Macromolecular Material: These systems are thermodynamically stable (reversible).

(3) Association Colloids (colloidal electrolytes): They are also thermodynamically stable.

Other classifications of colloidal systems have been based on chemical composition:inorganic (metals, non metals, oxides and hydroxide sols and colloidal salts), organic colloids (homopolar sols, heteropolar sols and hydroxy sols); according to particle shape : sphero (Globular) and linear (fibrous) colloids; according to structure: molecular and micellar (Jirgensons and Straumanis, 1964; Shaw, 1980).

Table 3.1: Colloidal dispersions
Source: Maron and Lando (1974), Shaw, (1980), Silcocks, (1982)

Dispersed phase	Disperse medium	Name	Example
L	G	liquid aerosol	fog,liquid sprays,mist, clouds
S	G	solid aerosol	smoke,dust
G	L	foam	foam on soap solution,fire extinguisher foam
L	L	emulsion	milk,mayonnaise,oil in water,water in oil
S	L	sol, colloidal suspension	paste,Au sol,AgI sol,S in H_2O
G	S	solid foam	expanded polystyrene, pumice stone
L	S	gel, solid emulsion	opal,pearl
S	S	solid suspension	pigmented plastics,glass coloured with dispersed metals

3.2 Surface Charge and Properties of Colloids

In water and wastewater engineering, colloidal suspensions and emulsions are the most important. Algae in water form colloidal suspensions. The properties of colloidal suspensions may be summarized as follows (Steel and McGhee, 1979; Barnes et al., 1981):

a. Colloids have large surface area per unit volume
b. Surfaces of colloidal particles are electrically charged
c. Colloidal particles follow a random walk due to Brownian motion.
d. Colloidal particles cause scattering of light (Tyndal effect).
e. Colloids cannot be removed by ordinary filtration without pretreatment

3.2.1 Origin of the Surface Charge

The charge on colloidal particles may be attained by the following mechanisms (Jirgensons and Straumanis, 1964; Lyklema, 1978; Bratby, 1980; and Shaw, 1980):

(1) dissociation (ionization): some chemical groups, which are constituents of the colloidal particles ionize into ions. For example (-COOH) or (NH_2) ionizes and renders the particle electrically charged. The material may be amphoteric and the charge is affected by pH. Bacteria gain their surface charge by this mechanism.

(2) uptake of ions from the solution: This process usually occurs by adsorption. The silanol group (SiOH) on the surface of silica particle may uptake H^+ which makes the surface positively charged; the surface gains negative charge by adsorption of OH^-.

(3) ion dissolution: ionic substances can acquire a surface charge by unequal dissolution of the oppositely charged ions of which they are composed; e.g. (AgI) depending on pH gains a negative charge when I^- exists in excess in the solution, and (AgI) particles acquire a positive charge when Ag^+ ions are in excess.

(4) isomorphic substitution: in the crystal lattice of clay, for example, Si^{4+} is exchanged by Al^{3+} or Ca^{2+}, which yields a net negative charge on the surface. This type is usually not affected by pH.

3.2.2 Surface Charge of Algae

Algal cells, except filamentous species, range in size between 5 and 50 μm (Tenney et al., 1969). This range is greater than the normally accepted colloidal size. Nevertheless, algal suspensions show colloidal properties. They have hydrophilic properties and in discrete form they make stable suspensions, possess negative charge and make chemical reactions on the surface (Ives, 1956, 1959).

Ives (1956) reported the presence and measured the negative charge density and zeta potential on algal cell surfaces. The size of algal cell did not affect zeta potential. The sign and magnitude of the primary charge of colloidal particles are affected by pH and ionic strength of the aqueous phase (O'Melia, 1972).

Microorganisms become positively charged at low pH because the amino groups NH_2 change to NH_3^+. At high pH values, negatively charged surfaces prevail due to dissociation of the functional carboxyl group COOH to COO^- (Kruyt and Overbeek, 1960; Bratby, 1980). For algae, Ives (1956,1959) found that negative values prevailed at all pH values investigated (2.5-11.5) and no isoelectric point was observed. Foess and Borchardt (1969) also found that algae remain negative for pH>1 but they also found that the isoelectric point occurred at pH=1. The negative charge increased with increase in pH due to adsorption of (OH^-) on cellulose of the cell wall. Lysis of chlorella cells and irreversible flocculation was noticed at pH 10.5.

A minimum zeta potential and charge density occurred at pH 7-8 for natural algae in ionized and ion-free water (Ives, 1956, 1959). Furthermore, Ives's work indicated the dependence of zeta potential and charge density on pH value is related to the ionic concentration of water. In ion-free water the charge density and zeta potential changed with pH. However, in natural waters the

relation was not obvious and the explanation offered was based on the masking effect of ions present in water on the change of H^+ and OH^-.

Experiments carried out by Golueke and Oswald (1965) revealed that a relationship existed between sedimentation of algae in distilled water and variation of pH by addition of H_2SO_4. It was observed that the most effective removal took place at pH=3. Inferior results were obtained at higher and lower pH values. Also, addition of 0-0.4 Moles of Ca^{++}, Mg^{++} or Na^+ had no effect on sedimentation.

3.3 The Double Layer Model

Since colloidal particles are electrically charged, and the whole colloidal dispersion is neutral, then there must exist counter charges - also called counter ions - to neutralize the charge on the surface of the disperse. Electrostatic coulombic attraction is highest when the distance between oppositely charged particles is smallest. Hence, the concentration of counter ions is maximum near the surface of colloidal particles. This concentration decreases with distance from the surface due to diffusion and decrease of electrostatic attraction (Fig. 3.1a).

Thus two layers can be distinguished in the double layer model: the charged surface of the particle and the diffuse layer which contains the counter ions. Later, the double layer was divided into three parts: a surface layer, a Stern layer and a diffuse layer (Fig. 3.2). The Stern layer is part of the solution side which is under the effect of the surface layer and it is held to the surface. It is usually not more than a few molecules thick. The diffuse layer (also called Gouy-layer) is part of the solution and it is within this layer the concentration of counter ions decreases with distance away from the particle surface (Overbeek, 1952-c; O'Melia, 1972; Lyklema, 1978; Bratby, 1980; and Shaw, 1980).

The potential at point x within an electric field is defined as the electric work needed to bring a unit charge of the same sign as the primary charge from infinity to that point x. This potential decreases with distance from the charged surface and is affected by the chemistry of solution (Fig. 3.1b and Fig. 3.2)

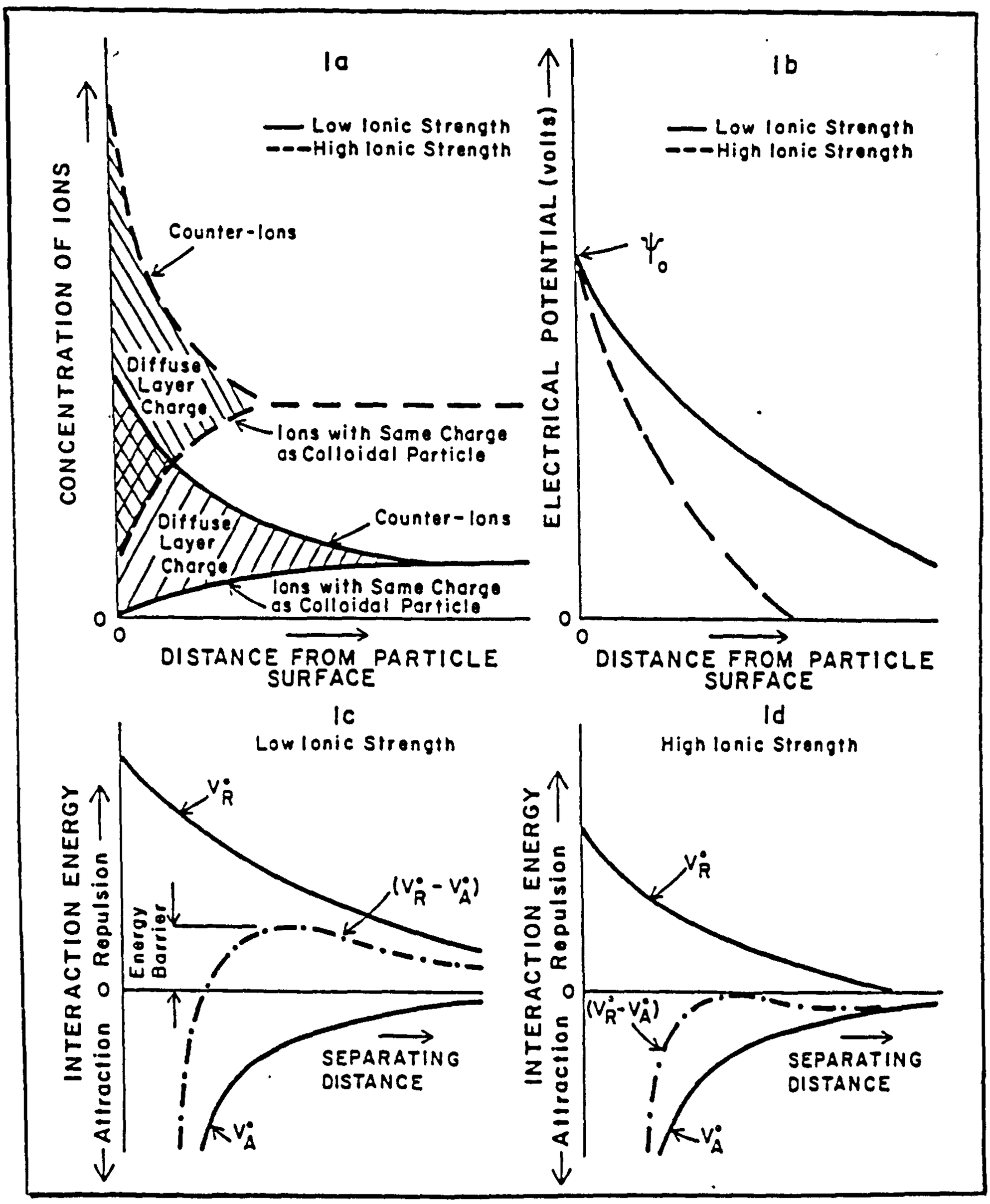


Fig.3.1: Schematic illustrations for a colloidal system:(3.1a) the electric double layer, (3.1b) the electric potential, (3.1c and 3.1d) the interaction energy
Source:O'Melia (1972)

This potential is determined experimentally by electrokinetic measurements (electrophoresis, electro-osmosis, streaming potential and others). When electrophoresis is used, the particle moves, with a thin layer of liquid moving with it, with respect to the rest of the stationary liquid. This layer forms a slipping plane which is usually considered to coincide with the outer border of the Stern layer. The electrokinetic potential measured at this plane is known as zeta potential, ζ (Lyklema, 1978) (see Fig. 3.2).

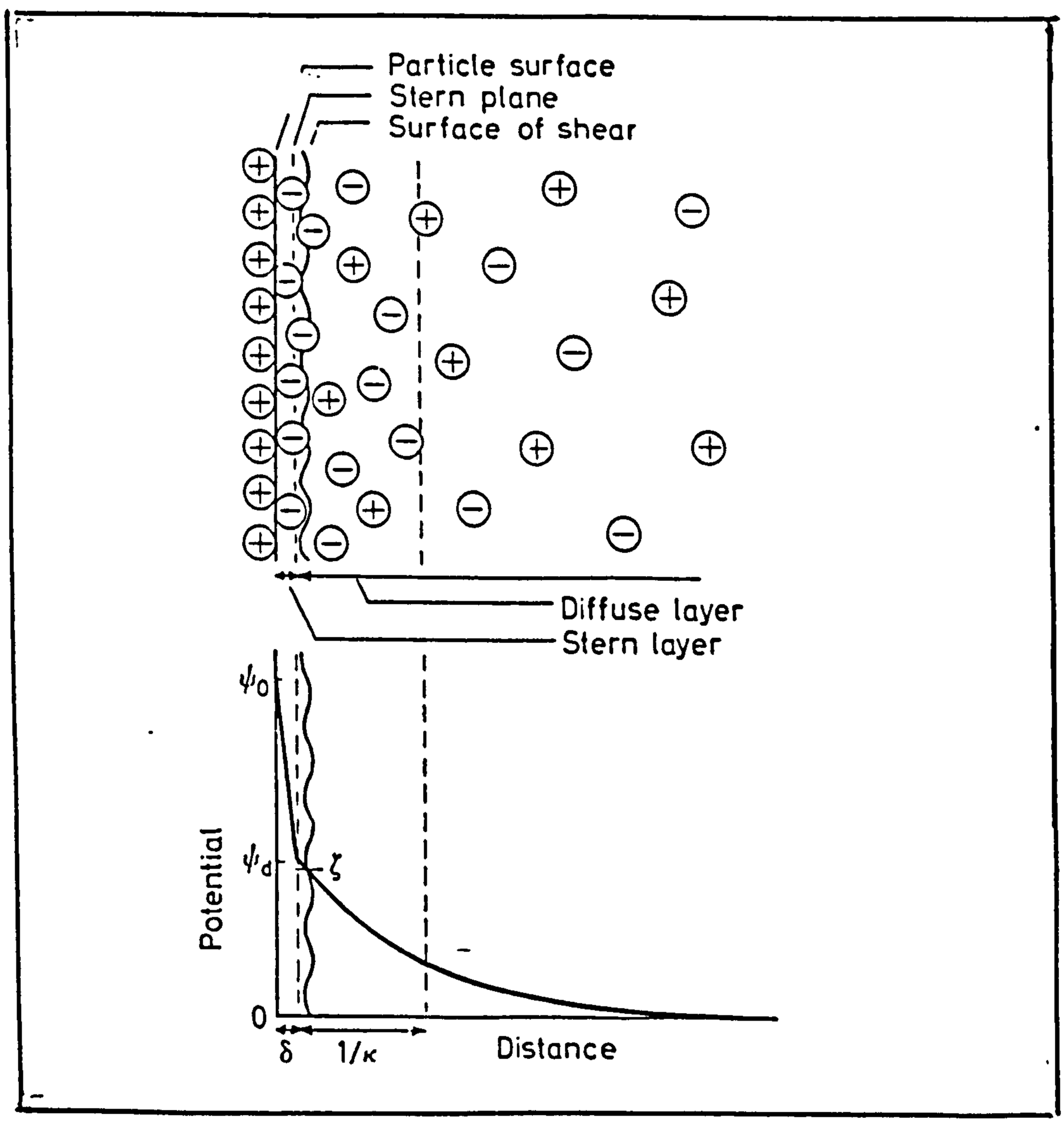


Fig.3.2: Schematic representation of the double layer model

Source: Shaw (1980)

3.4 Stability of Colloids

Colloids may be thermodynamically stable (reversible) or thermodynamically unstable (irreversible) (Kruyt, 1952).

A thermodynamically reversible process is one which can be caused to proceed in the opposite direction by making an infinitesimal change in the variables that control the system at any point in its progress (Glasstone and Lewis, 1970; Sheeham, 1970; Moncrief and Jones, 1977).

Thermodynamically stable colloids (reversible) can form spontaneously from their components. Lyophilic colloids belong to this category. On the other hand, lyophobic colloids are thermodynamically unstable (irreversible) systems that undergo reduction of the surface area by recrystallization, or agglomeration or coalescence (Van Olphen and Mysels, 1975).

Even in the absence of electrolytes, a lyophobic sol will form aggregates if given enough time. Hence, colloidal stability is a relative concept that can be expressed in terms of the aggregation rate (Van Olphen and Mysels, 1975). In water and wastewater engineering the stability of colloids is based on this time rate of aggregation concept. Stable colloids (diuturnal) are thermodynamically unstable ones that aggregate at a very small rate. On the other hand, thermodynamically unstable colloids that aggregate rapidly are described as unstable or caducous colloids (O'Melia, 1972).

Irreversible colloids owe their apparent stability to net repulsive forces that develop when two similar charges become closer to each other. Stability of lyophilic colloids is caused by solvation, in addition to repulsion of similar charges (Kruyt and Overbeek, 1960; O'Melia, 1972; Bratby, 1980).

3.4.1 Stability Due to Charge

Repulsion takes place when two particles of similar charges become close enough to each other such that the fields of influence of their surface charges coincide; or, in other words, their diffuse layers overlap. These repulsive forces

decrease with distance separating the two particles and their magnitude depends on the composition of the aqueous phase (Fig. 3.1c and 3.1d).

Colloidal particles are attracted to each other by van der Waals' forces. These forces are independent of the ionic strength of the solution.

The DLVO theory (Deryaguin and Landau; Verwey and Overbeek) quantifies these attractive and repulsive forces. The theory was explained by Hunter (1981).

The net effect of repulsive minus attractive forces was termed the potential barrier and sometimes energy hill or activation energy barrier (Overbeek, 1952b; O'Melia, 1972). This energy barrier causes the stability of hydrophobic colloids and must be overcome for aggregation to occur. The net effect depends on the ionic strength of the aqueous phase (Fig. 3.1c and 3.1d).

When the ionic strength is low, the effect of repulsive forces starts at a longer distance because the thickness of the diffuse layer is larger than in the case of high ionic strength of the aqueous phase (Fig. 3.1c and 3.1d). Also, zeta potential and surface charge of particles are decreased in the case of high ionic strength because more counter charges are adsorbed on the surface (Fig. 3.1b). Consequently, repulsive forces and stability of the colloidal systems decrease when the aqueous phase has a high ionic strength.

The kinetic energy supplied by the Brownian motion of colloidal particles counteracts the high energy barrier and reduces stability. Those colloids with a high energy barrier and/or low kinetic energy will aggregate very slowly (diuturnal). On the other hand, irreversible colloids that aggregate at a fast rate (caducous) will have a low activation energy barrier and/or high kinetic energy (O'Melia, 1972).

3.4.2 Solvation

Solvation is the binding of liquid on the surface of colloidal particles. When the dispersion media is water, solvation is known as hydration. In lyophilic colloids a layer of liquid surrounds the colloidal particles and prevents them from aggregation (Kruyt and Overbeek, 1960; Jirgensons and Straumanis, 1964).

The degree of solvation is dependent on the affinity of the solvent towards the atoms or functional groups forming the surface of particles. For example, H_3O^+ has no affinity towards CH_3 groups but it is held well to OH^- in starch (ibid).

The apparent stability of hydrophobic irreversible colloids is attributed to the charge of the particle and the energy barrier due to repulsive electrostatic forces. However, there are no strict hydrophobic colloids and all colloids are hydrated to a certain degree. Thus, many irreversible colloids are stabilized by hydration effects, in addition to charge effects (O'Melia, 1972). For hydrophilic colloids, stability is produced by both charge and solvation (Overbeek, 1952; O'Melia, 1978).

3.5 Destabilization of Colloids

Destabilization of colloids is changing them from low rate aggregating systems (diuturnal) to systems which undergo aggregation fast (caducous) (O'Melia,1972).

Many mechanisms may be involved in the process of destabilization, depending on the nature of colloidal system and coagulant added. These mechanisms include destabilization by double layer compression, electrostatic charge neutralization by adsorption of counter ions, physical enmeshment of fine solids in precipitating chemicals and interparticle bridging by adsorption of on polymer chains (EPA, 1975).

Destabilization is usually effected by addition of chemicals to colloids. When destabilization is caused by electrostatic effects only the added materials are called indifferent electrolytes (O'Melia, 1972).

3.5.1 Double Layer Compression

When an indifferent electrolyte is added to a colloidal suspension, the concentration of counter ions increases in the double layer. The thickness of the double layer is consequently compressed, because neutrality of the primary

charge is achieved by less volume at higher concentration of opposite charges. Since repulsion occurs when double layers overlap, compression of the double layer decreases the range of repulsive forces. Also, due to increased adsorption of counter ions, the surface charge and zeta potential of the colloids are decreased, which also shortens the distance within which repulsive forces are active. Consequently, the energy barrier is lessened, and attractive forces will be dominant at most distances. Hence, destabilization takes place (Van Olphen and Mysels, 1975; Bratby, 1980).

Different coagulants effect different degrees of destabilization. The higher the valence of the counter ion the more is its destabilizing effect and the less is the dose needed for coagulation. This is known as the Schulze-Hardy rule (Van Olphen and Mysels, 1975) and was quantified by the DLVO model relating the zeta potential to optimum coagulant dose (Hunter, 1981).

However, in water and wastewater engineering most coagulants are not indifferent electrolytes and undergo different interactions such as adsorption. Moreover, the stability of most colloids is affected to a certain degree by solvation (O'Melia, 1972).

In addition, the Schulze-Hardy rule is only applicable at low concentration of the electrolyte. At higher concentration, mechanisms other than double layer compression become involved (Bratby, 1980).

Optimum coagulant dose is not affected by colloid concentration in the case of indifferent electrolytes (Gregory, 1978; Bratby, 1980).

3.5.2 Adsorption and Charge Neutralization

Lower optimum doses of dodecylammonium ion, $C_{12}H_{25}NH_3^+$ than Na^+ ions were needed to bring turbidity to zero, although, both had a charge of (+1) (O'Melia, 1972; Bratby, 1980). Also it was noticed that restabilization occurred if excess coagulant was applied. These observations together with the fact that coagulant dose has often a stoichiometric relationship with particle concentration lead to the conclusion that mechanisms other than electrostatic forces are involved in coagulation.

In the amine the CH_2 group has no affinity to water and offers active sites for adsorption. Sodium ions, on the other hand, interact more strongly with water, which renders them not surface active. Charge neutralization then reversal and restabilization at higher doses of coagulant, such as alum and dodecylammonium ion, can be explained by adsorption of these ions.

3.5.3 Enmeshment in Precipitate:

It was observed by O'melia (1972) that when alum dose, $Al_2(SO_4)_3$, was low no flocculation occurs. Higher doses brings about destabilization and restabilization occurs if the dose of alum is increased. If the dose is further increased, removal occurs again. This last stage of removal of colloidal particles is caused by the formation and precipitation of $Al(OH)_3$. The colloidal particles are enmeshed in the precipitate. This enmeshment was termed "sweep floc" (ibid).

Enmeshment of colloidal particles in a precipitate can occur with other metal compounds, such as $FeCl_3$, CaO and $Ca(OH)_2$. Precipitation may be in the form of metal hydroxides (e.g. $Fe(OH)_3$ or $Mg(OH)_2$) or metal carbonate (e.g. $CaCO_3$).

3.5.4 Adsorption and Bridging

Anionic polymers have been successfully employed in coagulation processes, even for negatively charged colloidal particles. This cannot be explained by the charge effect alone. Destabilization is caused by links forming between polymer and colloidal particles when particles are adsorbed on the active sites of polymer (O'Melia, 1972; Barnes et al., 1981). If the polymer has the same sign as the dispersed particles, overdosing might cause restabilization due to over saturation of the active sites on the colloidal particles, which may prevent interparticle bridging (Hilson and Richards, 1980).

Polymers of higher molecular weight are more effective in bridge formation. The charge density on the polymer chains has a role in the degree of

destabilization. Increased charge density causes the polymer segments to expand, which increases the opportunity of adsorption. However, if the charges on particles have the same sign as that of the polymers, an increase in charge density decreases the tendency to adsorb (Gregory, 1978a).

3.6 Coagulants Used for Algae Removal

Many chemicals have been used to effect coagulation in water and wastewater treatment. They include aluminium sulphate, hydrated and quick lime, ferric chloride, ferric and ferrous sulphate and others. A description and characteristics of these chemicals may be found in many references (see for example AWWA, 1971a; Barnes et al., 1981).

Coagulation of algae has been carried out with alum (Golueke and Oswald, 1965; McGarry, 1970), with lime (Van Vuuren and Van Duuren, 1965; Folkman and Wachs, 1973), with ferric sulphate (Bare et al., 1975) and with polyelectrolytes (Tenney et al., 1969; McGarry, 1970). Their results will be presented briefly. Ayoub et al.(1986) tested seawater for algae removal.

3.6.1 Lime

Lime in the form of CaO or $Ca(OH)_2$ was found effective in coagulation of algae if pH was raised above 10.5 (Golueke and Oswald, 1965). Lime treatment is sensitive to pH level, but alum is less sensitive (EPA, 1975). Levels of pH 11.5-11.8 were found most effective for coagulation of algae by lime (Ayoub and Koopman, 1986). This high pH level needs to be lowered again after treatment, which adds to the cost and the processes needed. Besides, it was found that ($CaCO_3$) particles are always negative and did not cause algal flocculation, but partial sedimentation was observed (Folkman and Wachs, 1973). In addition to that, Graham and Hunsinger (undated) reported that batch lime treatment has been associated with inconsistent results for removal of phosphorus from wastewater stabilization ponds. Ronen (1981) found that lime treatment did not remove dissolved nitrogen compounds.

Golueke and Oswald (1965), also Oswald and Golueke (1968) carried out tests on the use of lime, ferrous sulphate and alum for algae harvesting. They found that alum was the most effective. Shindala and Stewart (1971) investigated the use of chemical treatment followed by sedimentation as a post process to remove algae and improve the quality of the effluent of stabilization ponds. They found that alum was most effective and superior to ferric coagulants on the basis of cost and because of the colour left by iron in the supernatant. Treating wastewater with lime requires high dosages of coagulant and produces large volumes of sludge. Hence, cost is increased (Van Vuuren et al., 1981; Leentvaar and Rebhun, 1982).

3.6.2 Polymers

Although the term "polyelectrolytes" refers to charged polymers, it is usually used to describe all polymeric coagulants possessing a charge or not (Gregory, 1978a). Inconsistent results were obtained by researchers regarding the suitability of polymers for algae removal from pond effluent.

Anionic and non ionic polymers did not effect algae removal from pond effluent (Tenney et al., 1969; Golueke and Oswald, 1965). Anionic cationic and non ionic polyelectrolytes did not improve the results of jar tests when added as a coagulant aid to alum for coagulation of algae in maturation pond effluent (Van Vuuren and Van Duuren, 1965). The same results were obtained by McGarry (1970) with anionic and nonionic polymers. Although cationic polymers improved removal when used as an aid to alum, they proved to be not effective when used alone.

Friedman et al. (1977) found that four cationic electrolytes tested where not effective in algae removal when used alone or in conjunction with lime. With alum as a primary coagulant only purifloc C-41 increased sedimentation. However, it was noticed that restabilization occurred in a small dose range (± 1mg/l) and it was recommended by the authors that extreme caution is required when this polymer is used.

Cationic polyelectrolyteswere found effective for algae removal at pH 2-4 depending on algae concentration. Higher concentrations of algae shift up the optimum pH (Tenney et al., 1969). Parker (1976) reported that good coagulation of algae could not be achieved at economic polymer doses.

3.6.3 Aluminium Sulphate (Alum)

The hydrolysis of alum produces acid according to Eq. 3.1. This requires that alkalinity should be present or added to react with the acidic product of hydrolysis so as to keep pH within the desirable range of coagulation. If alkalinity is high, excessive doses of alum will be required to reduce pH to the optimum (Barnes et al., 1981).

$$Al_2(SO_4)_3 + 6H_2O \rightleftharpoons Al(OH)_3\downarrow + 3H_2SO_4 \quad .... \quad (3.1)$$

In fact, Eq. 3.1 is a simplification of the situation and it is not always the case that aluminium sulphate reactions proceed to produce the electroneutral precipitate $Al(OH)_3$ (Bratby, 1980). When $Al_2(SO_4)_3$ is added to water at quantities less than the solubility limit of $Al(OH)_{3(s)}$, dissolution to $[Al(H_2O)_6]^{+3}$ occurs and as then followed by hydrolytic (hydrolysis) reactions that cause the formation of simple polymeric hydroxy metal complexes such as $[Al(H_2O)_5(OH)]^{2+}$ or $[Al(H_2O)_4OH_2]^{1+}$, depending on pH level. When more quantities of alum are added and the solubility limit is exceeded, the metal hydroxide precipitates. Also, complexes formed in hydrolysis combine together to form different polymeric molecules, including $[Al_6(OH)_{15}]^{3+}$, $[Al_7(OH)_{17}]^{4+}$, $[Al_8(OH)_{20}]^{4+}$, $[Al_{13}(OH)_{34}]^{5+}$ (Stumm and Morgan, 1962; O'Melia, 1972; Steel and McGhee, 1979). An approximate formula $[Al_x(OH)_{2.5x}]^{0.5x+}$ was set to describe these polymers by AWWA (1971).

The dosage of alum applied in practice usually exceeds the solubility limit of aluminium hydroxide. Thus, destabilization occurs by charge neutralization and adsorption and also bridge formation at intermediate concentrations of both

colloid and coagulant. Precipitation and enmeshment in the sweep floc also takes place at low colloidal and high coagulant concentrations (O'Melia, 1972).

Ives (1956, 1959) stated that attraction and charge neutralization between positive hydroxides of alum on algal cells are the main mechanisms of destabilization. He suggested that pH values on the acidic side, which keep the hydroxides of alum positive, give rise to optimum coagulation of algae by alum. Also, it was observed that the coagulant dose required increased with cell size, charge density and algae concentration. Enmeshment was considered a secondary cause that might contribute to the removal. Adsorption of algae on the surface of the floc did not seem to occur, since algae were found inside the flocs.

Field scale experiments for harvesting of algae from high rate pond effluent resulted in 94% removal of suspended solids at an alum dose of 105-120 mg/l (Golueke and Oswald, 1965; Oswald and Golueke, 1968).

Ford and Tischler (1976) reported the work of other investigators that alum was the most effective coagulant for algae removal. Dosages ranged between 250 and 300 mg/l. Alum was tested among other coagulants such as lime, ferric sulphate, cationic polyelectrolytes, and a combination of ferrous sulphate and lime.

Shindala and Stewart (1976) treated pond effluent by alum coagulation for removal of phosphates and COD.

3.7 Clay as a Coagulant

Bentonite has been utilized as a clarification agent for a long time. Bentonite clay has been used for clarification of wines (Saywell, 1935). Olin et al. (1942) reported that the use of bentonite in the treatment of domestic and industrial wastewaters improved the settling characteristics of particles and gave better quality effluent in terms of BOD and turbidity. In villages of the Sudan, people have used natural clay, which contains high a percentage of montmorillonite, for water coagulation (Jahn, 1976, 1977). In water treatment, clays have been used as primary coagulants or coagulant aids and as colour adsorbents (Cox, 1969; Packham, 1972; Schulz and Okun, 1984; Gersberg et al.,

1988). Doses of clay in the range of 10-50 mg/l as a coagulant aid lead to better floc formation, enhanced colour and organic matter removal, and broadened the pH range for effective coagulation (AWWA, 1971a). Longer filter runs and savings in the amounts of backwashing water, were obtained by use of bentonite as a coagulant aid in the treatment of water from lake Goy (Louis, 1956).

Clay offers a low cost alternative as a coagulant for algae removal from facultative pond effluent. A summary of its characteristics is introduced in the following sections.

3.7.1 Types and Composition of Clay

Clay refers to the faction of soil of less than 2 μm effective diameter which has cohesive properties (Wihun and Starzewski, 1975; ; Bowles, 1982, 1984; Tan, 1982). Mineralogists and soil chemists use the term "clay mineral" to refer to minerals with sheet silicate structures (phyllosilicates) because the clay-sized fraction of many soils consists largely of these layer-silicates, plus small amounts of other crystalline and amorphous materials (Marshal, 1964; Deer et al., 1966; Bohn et al., 1985).

A typical clay mineral is composed of two types of structural units. The first is a tetrahedral array of $(SiO_4)^{4-}$ and the second is an array composed of cations (such as Al^{3+}, Mg^{2+}, Fe^{3+}, Fe^{3+}) in octahedral coordination with oxygen or hydroxyl anions (Van Olphen, 1963; Tan, 1982; Bohn et al., 1985).

Clay minerals differ according to their inter-layer cations and the nature of bonds between layers. Chemical composition and surface charge vary according to the extent of replacement of Si, Al, Mg by other cations (Deer et al., 1966; Bohn et al., 1985).

According to Deer et al., (1966) and Tan (1982) clay minerals may be divided into the following main groups:

1. Kandite group, including kaolinite, dickite and nacrite, halloysite and meta-halloysite.

2. Illite group (hydrous mica; soil mica), including illite, hydro-micas, phengite, brammallite, glauconite and celadonite.
3. Smectite group, including montmorillonite, nontronite, hectorite, saponite, sauconite, and beidellite.
4. Vermiculite
5. Palygorskite group, including palygorskite, attapulgite, and sepiolite.

The two minerals that are usually used in water and wastewater treatment are kaolinite and montmorillonite. These two minerals will be briefly reviewed in the following sections. Bentonite, a clay that is usually mentioned in water treatment, is a natural clay deposit that contains large amount of montmorillonite; it is the commercial grade of montmorillonite (Grim and Güven, 1978; Tan, 1982).

(I) Kaolinite

Kaolinite has a general composition of $Al_2Si_2O_5(OH)_4$ or, in terms of oxides, $Al_2O_3.\ 2SiO_2.\ 2H_2O$ (Deer et al., 1966; Brown et al., 1978; Bohn et al., 1985).

A unit layer of Kaolinite is composed of two sheets. A tetrahedral silica sheet is bonded to a gibbsite sheet by oxygen atoms that are mutually shared by silicon and aluminium atoms in the respective sheets. The unit layers are held together by a hydrogen bond between OH^- ions in the octahedral sheet in one layer and the oxygen atoms in the tetrahedral sheet in the second layer (Tan, 1982) (Fig. 3.3a).

The strong hydrogen bond makes the interlayer spacing too small for cations to enter and little isomorphic substitution occurs. Also, the exposed surface area of the mineral is relatively low (7-30 m^2/g), which limits Kaolinite's adsorption and cation exchange capacity. However, due to the exposed OH^- group, Kaolinite still possesses a pH-dependent negative charge and a small cation exchange capacity, usually in the range of 1 to 10 meq/100g (Van Olphen, 1963; Tan, 1982; Bohn et al., 1985).

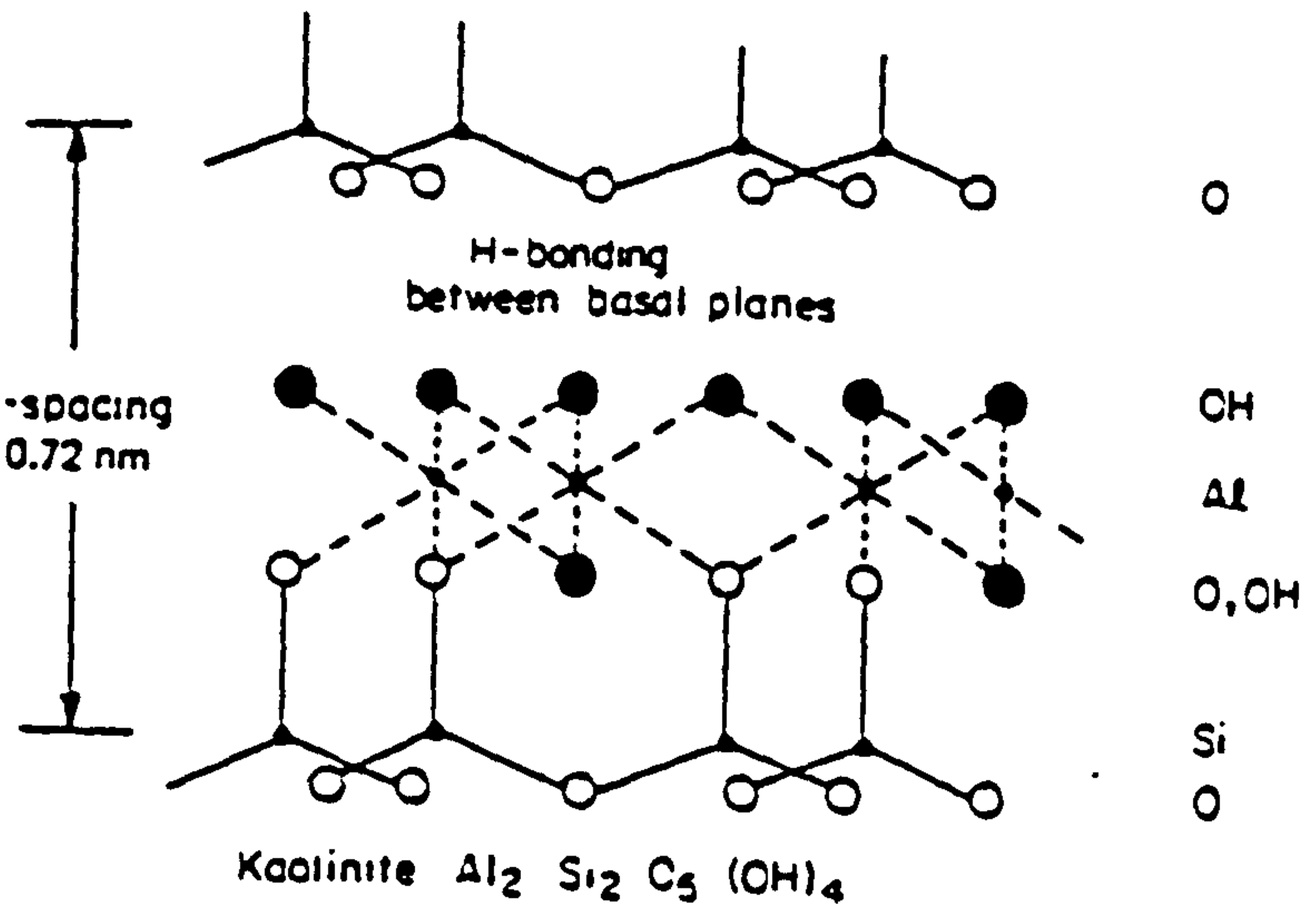


Fig. 3.3: (a)Schematic structure of kaolinite (b)Schematic structure of bentonite

Source: Bohn et al. (1985)

(II)Montmorillonite

The general formula of montmorillonite may be written as $Na_{0.7}(Al_{3.3}Mg_{0.7})Si_8O_{20}(OH)_4.nH_2O$. Its basic layer consists of an octahedral aluminium sheet (gibbsite $Al_2(OH)_6$) sandwiched between two tetrahedral silica sheets (SiO_4^{4-}) (Fig. 3.3b). Layers are held together by weak van der Waals' forces which causes inter layer swelling, large specific surface area (700-800 m^2 /g), large cation exchange capacity 80-150 meq/100g and adsorption not only on the external surfaces of the particles but also between the unit layers. The surface charge is mainly due to isomorphic substitution and is not pH-dependent (Deer et al., 1966; Tan, 1982; Bohn et al., 1985).

3.7.2 Charge and Colloidal Chemistry of Clay

Clay particles usually have a charge on their surfaces. This charge develops mainly by dissociation of exposed hydroxyl groups and adsorption of protons, as was previously explained for colloids in general. In addition to these mechanisms, isomorphic (amorphous) substitution is a major cause of charged surfaces in some types of clay, such as montmorillonite. Due to surface charge, an electric double layer is formed (Tan, 1982).

Depending on the origin of the charge, the value of pH may or may not affect this charge. Charges produced by amorphous substitution are generally not affected by pH value. On the other hand, the pH level will affect the sign and intensity of charge and an isoelectric point will be noticed if the charge originated due to dissociation of exposed (OH) groups or adsorption of protons.

Isomorphic substitution occurs when the silica in the tetrahedral sheet is replaced by ions of similar size and lower valency, usually Al^{3+}. Also, in the octahedral sheet part of (Al^{3+}) may be replaced by Mg^{2+} leaving a permanent charge without disturbing the crystal structure. Isomorphic substitution occurs between ions whose size does not differ by more than 10%-15% and the valency difference should not be more than one unit (Tan, 1982; Bohn et al., 1985).

Kaolinite has an exposed OH group on the surfaces and edges of its layers. The edge group is Aluminium (Al-OH) which is an amphoteric group that becomes positively charged if pH is under the isoelectric point and negatively charged if pH value is above the isoelectric point (Tan, 1982):

$$\text{-Al - OH} + \text{OH}^- \rightleftharpoons \text{-Al -O}^- + \text{H}_2\text{O} \quad \text{(alkaline medium)..(3.2)}$$

$$\text{-Al - OH} + \text{H}^+ \rightleftharpoons \text{-Al - OH}_2^+ \quad \text{(acidic medium) ..(3.3)}$$

The isoelectric point at the edge of kaolinite was reported to occur at pH 6.3 (Tan, 1982) and at pH=8 (Greenland and Hayes, 1978). Dixon (1977) reportcd that some workers found an isoelectric point at the edge of kaolinite at pH=4 and others at pH=7.3.

The Silanol group, SiOH, on the surface of kaolinite dissociates to Si-O$^-$. This group does not accept a proton above pH=2. It is not amphoteric in the pH range 3-to-10 (Greenland and Hyes, 1978). The surface charge on the Kaolinite surface is permanent. However, variation could occur to this charge by adsorption of silicates anion on the gibbsite surface (Greenland and Mott, 1978).

Montmorillonite layers were also reported to have aluminol groups at their edges and a positive charge is attained at pH=9. However, this charge is small and may reach a maximum density of 4 meq/l at pH=3 (Greenland and Mott, 1978). Van Olphen (1973) and Borchardt (1977) reported that this positive charge at the broken edges of montmorillonite depends on pH and an isoelectric point was observed at pH around 7.

Due to the existence of charges on clay surfaces and edges, and their relatively large specific area, several chemical activities take place among clays and other constituents of the solution. Adsorption, cation exchange and anion exchange all take place with clays.

(I) Adsorption:

Mechanisms responsible for adsorption on the clay surface include physical forces, hydrogen bonding, electrostatic bonding, and coordination

bonding (Harter, 1977; Tan, 1982):

(1) **Physical forces:** van der Waals' forces are the most important. These are weak electrical forces resulting from oscillation of charges in the atoms which gives rise to the formation of dipoles that attract each other. These forces only work at a short distance but have an additive property.

(2) **Hydrogen bonding:** water dipole is adsorbed on clay surfaces through linkage with hydrogen bonding between hydrogen of the water molecule and oxygen on the surface of the clay particle. There are many types of hydrogen bonding that take place with different substances.

(a) Protonation at the mineral surface: This occurs when an uncharged molecule approaches an acid mineral surface. The carbonyl groups and amine groups of proteins and peptides may be protonated in acid clay systems and consequently bonded by electrostatic forces to the surface of the clay.

(b) Water bridging: The water molecule is attracted to a cation on the clay surface and the H of the water molecules shares electrons with organic molecules.

(c) Bonding to mineral hydroxyl group: OH groups are exposed only at the edge of montmorillonite, but they form 50% of the Kaolinite surface and provide potential sites for hydrogen bonding with organic matter as an example.

(d) Surface oxygens of clays make hydrogen bonding with the H in organic matter.

(3) **Electrostatic bonding:** many organic matters are positively charged and are adsorbed on negatively charged surfaces of clay.

(4) Coordination reaction: mainly occurs with organic matter by sharing electron pairs with a metal on the clay surface. This type of reaction allows negatively charged molecules to be bonded to clay mineral surfaces.

(II) Cation Exchange

Previously adsorbed cations on the surface of clay may be replaced by other cations, depending on their degree of bonding to the surface. Generally, ions of higher valency are more strongly adsorbed. Also, ions of smaller hydrated radius are more preferred for adsorption. Ions of smaller dehydrated radius have a greater density of charge per unit volume, which makes them attract more water molecules and attain larger hydrated radius. The following adsorption preference was reported (Tan, 1982; Bohn et al., 1985):

$$Th^{4+} > Al^{3+} \approx La^{3+} > Ba^{2+} \approx Sr^{2+} > Ca^{2+} > Mg^{2+} \approx C_s^{+} > Rb^{+} > NH_4^{+} \approx K^{+} > Li^{+}$$

Examples of the cation exchange reactions that might take place are :

$$3/2\ Ca(HCO_3)_2 + Al\text{ - clay} \rightleftarrows Ca_{3/2}\text{-clay} + Al(OH)_3 + 3CO_2 \quad ...(3.4)$$

$$Ca\text{ - clay} + 2NH_4^{+} \rightleftarrows (NH_4)_2\text{ - clay} + Ca^{2+} \quad ..(3.5)$$

Under certain conditions some cations may enter between expandable layers of clay and become trapped and fixed there. Other exchanging cations may not be able to enter the space because of their large size.

The ability of clay to adsorb and exchange cations is called cation exchange capacity (CEC) and usually expressed in meq/100g of clay.

Cation exchange reaction are almost instantaneous at the exchange site but sometimes are limited by the diffusion rate of cations through thick adsorbed water layers. These exchange reactions are chemically equivalent or stoichiometric and are reversible. Because of their reversibility, these reactions

can be forced to proceed in either direction by changing the concentration of reactions or products.

(III) Anion Exchange

Two types of anion adsorption onto clay particles may take place: positive adsorption on positively charged surfaces or edges of clay, and negative adsorption on surfaces possessing a negative charge.

Previously adsorbed anions may be exchanged by other anions, but anion exchange capacity for clays is less than CEC. The lyotropic series of positive adsorption of anions was given by Tan (1982) in decreasing order :

$$SiO_4^{4-} > Po_4^{3-} >> SO_4^{2-} > NO_3^- \approx Cl^-$$

An example of positive adsorption is the adsorption of phosphates on positively charged clay.

$$Al\text{-}OH\text{-}clay + H_2PO_4^- \rightleftarrows Al - H_2PO_4 + OH^- \quad ...(3.6)$$

(IV) Coagulation

Kaolinite's positive charge on the edges at pH 3-8, and the pH-dependent negative charge on the surface causes flocculation of clays by edge to face interaction, especially when both charges are at their mid values in the pH range 4-5 (Greenland and Mott, 1978; McCooke and West, 1978).

The low negative charge of Kaolinite increases its tendency to flocculation. Face to face coagulation of Kaolinite is induced by salts that accompany the mineral, especially by trivalent and divalent cations such as Al^{3+} and Ca^{2+} which compress the double layer and overcome repulsive forces. Solids with excess Na^+ will show stability (Dixon, 1977; Gast, 1977). However, at high pH values two opposing mechanisms are experienced: first, increase of negative

charge density which causes dispersion; second, dissolution of aluminosilicates which oppose dispersion (Dixon, 1977).

In the presence of bicarbonate alkalinity, Al^{3+} may be exchanged by Ca^{2+} to produce $Al(OH)_3$ which has a coagulation effect (Tan, 1982). Also, Kaolinite reacts with water and produces gibbsite (Al_2O_3), with protons in acidic medium and produces Al^{3+}, $AlOH^{2+}$, $Al(OH)_2^+$ complexes (Marshal, 1977) that are all act as coagulants.

Bentonite forms two types of double layers in water suspensions. The first is associated with the surface; the second surrounds the edges of clay particles.

The isoelectric point for the edges occurs around pH=7. Thus, in acidic and neutral media an edge to face interlocking will take place. Face to face aggregation is effected by van der Waal's forces and is independent of pH but the degree of interaction is increased with the increase in concentration of bentonite (Van Olphen, 1973).

3.8 Factors Affecting Coagulation

It was earlier mentioned that hydrophilic colloids are more stable than hydrophobic ones. Thus the former need higher coagulant doses for destabilization than the latter. Also, it was explained how pH affects the charge sign and density of clays. For Aluminium hydroxide also, positive species of hydrolytic products such as $Al(H_2O)_5(OH)^{2+}$ are formed at pH below the isoelectric point (pH=9). Negative complexes,such as $Al(OH)_4^-$, will be produced above the isoelectric point (Letterman and Sricharoenchaikit , 1982). Increased ionic strength compresses the diffuse layer and decreases stability (Langelier et al., 1952; Van Olphen, 1963; O'Melia, 1972). Other factors that may affect the process of coagulation include temperature, concentration of both colloidal particles and coagulant, and mixing and flocculation conditions (Bratby, 1980; Kuo et al., 1988).

3.8.1 Temperature

Different results were obtained by researchers concerning the effect of temperature on coagulation. Cleasby (1984) concluded that temperature does not affect the coagulation process although it might affect settling velocity. He argued that Brownian flocculation (perikinetic) that is affected by temperature takes place within 6-10 seconds and this time of temperature-dependence can be ignored if compared with total flocculation time. He also reported other workers' findings that the flocculation process is independent of temperature.

On the other hand, Culp and Culp (1974) reported that coagulation is affected by temperature. Camp and Conklin (1970) found that higher temperatures produced higher floc volume concentrations. Also, Hudson and Wagner (1980, 1981) found that floc settling deteriorated after flocculation at 4°C for 30 minutes at mean velocity gradient (G) of $11s^{-1}$. At 21°C, no impairment of floc settling was experienced after prolonged flocculation of 40 minutes with $G=14\ s^{-1}$.

Slight improvement in algae removal by alum coagulation-settling was noticed by Golueke and Oswald (1965) and Al-Layla and Middlebrooks (1975) at 20°C and 10°C compared with 25°C, 30°C and 40°C. The explanation offered was that at higher temperatures the solubility of gases decrease. Gases leaving the dispersion medium during agitation are trapped within flocs being formed and produce less settleable flocs, although the decrease in viscosity at high temperature should have improved settling.

3.8.2 Concentration of Colloid and Coagulant

At low concentration of colloidal particles, high doses of coagulants are needed to effect destabilization because the chance of collision between particles is low and removal occurs by sweep floc. In this region, the coagulant dose decreases with increase in concentration (O'Melia, 1972). Coagulant aids such as silica and clays are added in this case to increase the opportunity of contact

and to improve floc settleability (Packham, 1972; Benedeck and Bancsi, 1976; Bratby, 1980).

If the concentration of a colloid is higher, and destabilization is caused by charge adsorption and neutralization, less coagulant dosage is required but the higher the concentration of colloid generally the higher the dose needed in stoichiometric ratios. In this region, restabilization may occur at higher coagulant doses.

If the concentration of colloids is still increased, then higher doses of coagulant are needed to neutralize charges but restabilization does not take place easily (O'Melia, 1972).

If the dose of coagulant is increased above the dose which brings about restabilization, then precipitation of aluminium hydroxide causes removal of turbidity by sweep floc (O'Melia, 1972; Letterman and Vanderbook, 1983).

Generally,, if a low coagulant dose is added, whatever the concentration of colloid, effective coagulation does not take place and only partial removal of turbidity is achieved. Too much coagulant dose may cause restabilization, depending on type of coagulant and particles and the mechanisms of destabilization (Langelier et al., 1952; O'Melia, 1972).

3.8.3 Mixing Time and Intensity

Mixing prior to flocculation helps to disperse the coagulant uniformly and increases the chance of floc formation (Hudson, 1965; Amirtharajah, 1978a; O'Melia, 1978; Twort et al., 1985).

The degree of turbulence required for mixing is usually expressed by the mean velocity gradient, G (Camp and Stein, 1943).

$$G = \sqrt{\frac{P}{\mu V}} \quad .... \quad (3.7)$$

G = mean velocity gradient, s^{-1}

P = power dissipated in the liquid, N.m/s

μ = viscosity, $N.s/m^2$

V = volume, m^3

The value of G in rapid mixing was shown to affect the coagulation clarification process. High values of G produced smaller flocs and delay of floc formation during flocculation (Camp, 1968; Camp and Conklin, 1970; Stevenson, 1980).

A long time of rapid mix is not required for hydrolytic reactions, which are instantaneous (Hudson and Wolfner, 1967; O'Melia, 1978; Amirtharajah, 1978a). Griffith and Williams (1972) found that turbidity removal after flocculation and settling was not influenced by rapid mix detention time from 5-to-60 seconds. Similar findings were reported by Benedeck and Bancsi (1976) and a maximum of one minute was recommended.

However, Letterman et al. (1973) found an optimum rapid mixing period associated with minimum remaining turbidity for coagulation by alum. An empirical model was introduced relating optimum mixing time with mean velocity gradient.

3.8.4 Time and Velocity Gradient during Flocculation

Camp and Stein (1943) and Camp (1946) found that the time rate of flocculation caused by the motion of fluid at a point in the fluid is directly proportional to the absolute velocity gradient at that point, which can be substituted by a mean velocity gradient G throughout the tank. This means that flocculation is affected by time of flocculation, t in addition to G. Camp (1955) proposed the product Gt as a dimensionless number to judge the adequacy of flocculation. He reviewed 20 plants in the USA and found flocculation basins were operating satisfactorily with Gt=23000 to 210,000 (G 20s^{-} -74s^{-1}) and concluded that higher G values of flocculation will produce small non-settleable solids. Also, Camp (1968) noticed that high G values decreased floc size at higher values of G. Huck and Murphy (1978) identified floc breakage at high values of G.

However, Tambo and Watanabe (1979) found that floc density decreased and settling velocity linearly decreased as floc size increased. Also, Hudson

(1965) found that settling velocity was not affected by high velocity gradients because the small particles produced at high G values had greater density.

Hemenway and Keshavan (1968) found that 30 minutes of flocculation achieved better removal than 15 minutes flocculation with polyelectrolytes. Ogedengbe (1976) found that flocculation time improved the polyelectrolyte coagulation-clarificationprocess when increased from 2-to-24 minutes. After that, either deterioration of performance took place, or no significant improvement could be obtained.

3.9 Summary

The previous literature review indicates that alum has been the most successful coagulant for algae removal. However, the high cost of coagulant may hinder its use. Thus, experiments in this work were carried out to define the effect of settling time and/or addition of coagulant aids on reduction of alum dose required for algae removal.

Kaolinite and bentonite have performed well as clarifying agents. Also, these clays have been effective in the removal of bacteria and viruses. In view of their low cost, these two types of clay were tested as primary coagulants and as coagulant aids for removal of algae.

The jar test was used for the assessment of these alternatives for algae removal. The test was designed such that comparison between different coagulants was possible. Also, the effect of sedimentation time was incorporated into the procedure of the test.

CHAPTER 4

Materials and Methods

4.1 Rock Filter Experimental Design

In the previous Chapter the following variables were considered to affect the performance of rock filters : rock size of filter media, height of media, hydraulic loading, concentration of TSS in the influent, algal species and water temperature. The design of the experimental unit for Al-Samra wastewater stabilization ponds (SWWSP) in Jordan took all these variable into consideration.

Al-Samra ponds consist of three almost identical trains, each train being composed of ten ponds. Influent to the filters was siphoned off 50 cm below the water level of pond M3-4, i.e.the last pond in train 3.

4.1.1 Rock Size and Height of Filter Media

Table (2.2) shows the different sizes of rock used in existing rock filters (0.47-20 cm). In Illinois, difficulties were faced with rock sizes smaller than 7.6-15 cm in the horizontal flow mode. O'Brien (1976) reported that clogging occurred after about one year of operation of a horizontal flow filter with rock ranging in diameter from 0.47-2.54 cm, with the majority greater than 1.27 cm. He recommended rock sizes between 2.54 and 12.7 cm, with most around 5.0 cm. Based on this, and for the purpose of this project, 5.0 cm rock size was chosen. Clogging is not a critical problem in vertical flow because the filter can be backwashed but, in order to study the efficiency of removal by a smaller rock size, another rock filter containing rock of 1 cm average diameter, was constructed. A third rock filter with 10 cm rock diameter was also built to study the higher range of rock size in the upflow mode.

The rock used to fill each filter was unisize. Two sieves were used for screening the required size in each filter. Rock that passed the upper sieve and was retained on the lower was used.

To consider the effect of height of filter media, different approaches can be utilised : a shallow filter can be used and results can be extrapolated by filtration theory; the same filter can be operated at different depths for each test run; many filters of different heights can be operated in parallel; and a deep filter with sampling points at different heights may also be employed (Ives,1966).

The last approach was chosen for the purpose of this research. The three filters were built with an effective height of 2 metres. Sampling points were installed at 50, 100, 150, and 200 cm from the filter bottom. The top of each filter was covered with an extra 20 cm of rock to prevent growth of algae on the surface (Fig. 4.1).

4.1.2 Hydraulic Loading

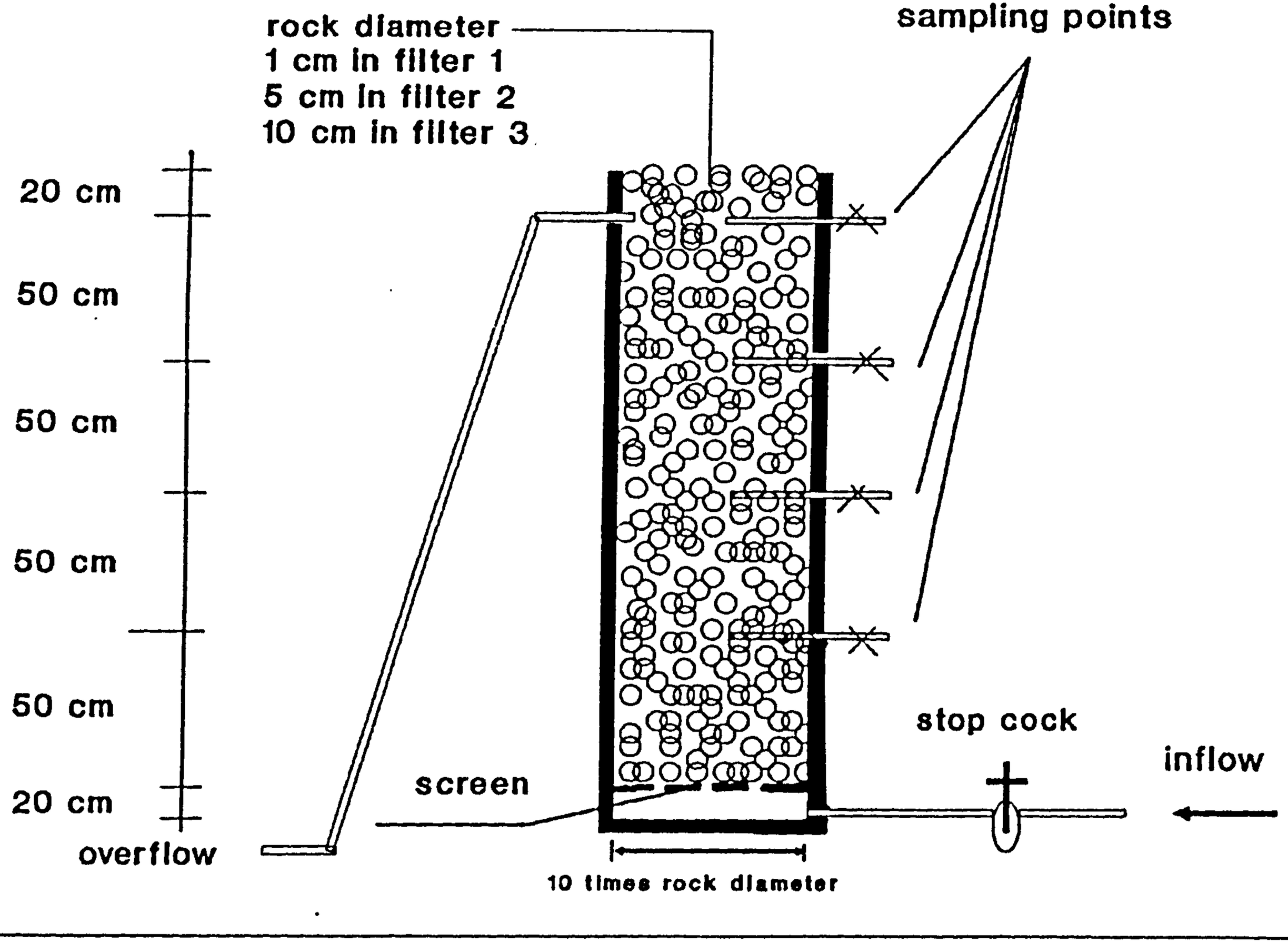
Table (2.2) shows that the hydraulic loading adopted in rock filtration has ranged between 0.06 and 2.2 $m^3/m^3.d$. At any one location the range applied is usually small and does not help to establish a relationship between hydraulic loading and performance.

For the purpose of this work, a composite design of two factors was chosen. Thus, taking hydraulic loading as one of the two factors requires that five levels of hydraulic loading should be tested. The experiments were run at the following hydraulic loadings: 0.1, 0.5, 1.0, 1.5, and 2.0 $m^3/m^3.d$. Values less than 0.1 $m^3/m^3.d$ are impractical. Also, rates higher than 2.0 $m^3/m^3.d$ were not recommended in the literature.

4.1.3 Algal Concentration and Species

Examining previous data of the effluent of SWWSP reveals that TSS is changing throughout the year. The pilot scale work was divided into three periods to define the effects of algal concentration. During each period, tests

Fig. 4.1:Schematic diagram of the rock filter



were run applying the five hydraulic loadings mentioned in the previous Section. Experimental set No. 1 was applied in December 1990, January and February 1991. Experimental set No. 2 was applied in March and April 1991. Set No. 3 was applied in May, June, July and August 1991.

Algal concentration, algal species and temperature were uncontrollable variables in the field experiments. They were tested during the three experimental sets.

4.1.4. Pilot Plant Dimensions

The use of models to simulate systems where water movement is considered has to ensure dynamic and geometric similarity between the model and the full scale. To achieve geometric similarity the dimensions of the model are kept at a constant ratio to the corresponding dimensions of the prototype. Dynamic similarity is established by maintaining the same ratio of significant forces in both systems: the ratio of inertia force to fluid viscous force (Reynolds Number), and the ratio of fluid inertia force to fluid gravity force (Froude Number).

Some physicochemical forces are involved in attachment mechanisms in the filtration process (Gregory,1975). However, Ives (1986) stated that these forces are almost impossible to scale up and only geometric similarity should be considered. Geometric similarity is achieved if the same media size is used in both the model and the full-scale unit.

The resistance offered to fluid flow is due to both the surface area of walls of the container and the media. The wall of the container affects the flow in two ways: the zero velocity at the wall affects the average velocity compared with flow in an infinite cross section. The second and main effect in packed columns arises from the increase of porosity near the wall, because of lack of interlocking, which also affects the average velocity (Dudgeon, 1967; Somerton and Wood, 1988).

At full scale, the effect of the wall is relatively negligible. To minimize wall effects in the model, the ratio of the specific surface of the walls of the

column to the specific surface of the filter aggregate should be kept as small as practicable (Sinkoff et al., 1959).

Rose (1945) and Rose and Risk (1949) concluded that a ratio of column diameter to material diameter (D/d_c) greater than 50 would entirely eliminate the wall effect. James (1989), after reviewing the paper of Rose, said that a ratio of 10 is a reasonable compromise. In fact Rose (1945), used a (D/d_c) ratio of 8 in carrying out his experiments.

Also, Sinkoff et al. (1959) considered that $D/d_c = 8$ would leave wall effects negligible. ANSI/ASTM (1980) adopted a ratio of 8 or 12 for the purpose of the permeability test.

It is not only required to minimize the wall effect but, in addition, the effect should be kept constant from column to column. This means that the same container diameter to pack diameter ratio should be kept constant in different reactors (Sinkoff et al., 1959).

A tubular shape of model filter and a media height not less than one metre, and preferably 1.5 - 2.0 m, was recommended by Ives (1966, 1986). Hence, three cylindrical filters of 2 m media height were constructed with a ratio of $D/d_c = 10$. The first was 10 cm diameter and contained 1.0 cm diameter rock media. The second was 50 cm and the third was 100 cm diameter, packed with 5 cm and 10 cm rock diameter, respectively.

4.1.5. Composite Design

Three rock filters with different rock diameters were built to study the effect of rock size. The effect of depth was considered by taking samples from different heights. However, since the TSS concentration in the influent changed continuously throughout the run period, a composite design for hydraulic loading and TSS was arranged.

A composite design of two factors requires $2^k + (2k+1)$ factor combinations, where k is the number of factors. Nine factor combinations are required for two factors (hydraulic loading and influent TSS concentrations) each

set at five levels in order to fit the full second order model (Deming and Morgan, 1988):

$$Y_{ii} = \beta_0 + \beta_1 x_{1i} + \beta_2 x_{2i} + \beta_{11} x_{1i}^2 + \beta_{22} x_{2i}^2 + \beta_{12} x_{1i} x_{2i} + x_{1i} \quad \ldots (4.1)$$

However, in order to determine purely experimental uncertainty, at least three replicates should be carried out at the central point.

By coding the five levels of the two factors x_1 and x_2 with -2,-1,0,+1,and +2, Table (4.1) shows the required factor combinations. The first four combinations are due to factorial design and the last five are from star design.

Table 4.1: Two factor combination for central composite design.

x_1	-1	-1	+1	+1	0	-2	+2	0	0
x_2	-1	+1	-1	+1	0	0	0	-2	+2

4.1.6 Parameters Measured and Frequency of Testing.

Parameters measured included TSS, VSS, turbidity, chlorophyll-a, total and soluble BOD_5, total and soluble COD, total nitrogen, ammonia, total and soluble phosphorus, temperature, pH, DO, and algal species.

As mentioned earlier, the work was divided into three experimental sets. In each set the five hydraulic loadings were applied. Each application is considered as one run of the test. In set No. 1 each run took two weeks. One week and three weeks were the lengths of runs in sets 2 and 3, successively. In set 1, the filters were backwashed after each run. No backwashing was carried out in sets 2 and 3.

Total suspended solids, turbidity and chlorophyll-a were measured in the influent to the rock filters and at all sampling points. The rest of the parameters

were measured only in the inflow and at the height of 2.0 m, which represents the effluent.

Grab samples were taken around 10 a.m three times a week. Algal species were identified monthly. Those grab samples were analyzed for TSS, VSS, turbidity, DO, PH, temperature, and chlorophyll-a. BOD, COD, Phosphorus and Nitrogen tests were only performed once a week.

4.2 Jar Test

Jar test is the term used to describe a set of tests that help to assess the process of coagulation-clarification of water or wastewater (Lockyear and Stevenson, 1986).

Although there are models that describe the process of flocculation (see, for example, O'Melia, 1972; Bratby, 1980), these models cannot be used for predicting optimal coagulation dosages or optimum pH (Alaerts and Van Haute, 1982). This insufficiency of models may be attributed to a variety of reasons: some parameters in these models are difficult, and others even almost impossible, to determine (O'Melia, 1972, 1978); the characteristics of water or wastewater to be treated vary widely, which necessitates use of trial as a means of selecting coagulant and determining the optimum dose (Fair et al., 1968); the models do not incorporate all parameters that affect the coagulation process; in addition the complexity of the turbulent field during the process makes jar tests better employed to determine design or operational parameters (Amirtharajah, 1978a; Stevenson, 1980).

Basically, in the jar test water or wastewater to be treated is placed in containers, coagulant is added at different doses, mixed (flash mix) and flocculated at predetermined times and speeds. Then the supernatant is tested for certain characteristics, such as turbidity or filterability, and the optimum dose is determined.

4.2.1 Types of Jar Test

Neither the procedure of the jar test, nor the equipment are standardized. The procedure described by AWWA (1982a) utilized squares jars. Cylindrical jars were referred to by other authors (Stevenson, 1980; Lockyear and Stevenson, 1986). Two versions of the settling jar test were described by Lockyear and Stevenson (1986): the first utilises 200 ml samples and mixing paddles 50 mm wide and 16 mm high. The second involves 800 ml samples in 1 litre beakers and the mixer is a four bladed paddle 50 mm in diameter and 18 mm height.

Jackson and Sheiham (1981) described three types of the jar test used by the Water Research Centre (WRC/UK):

(1) Sedimentation Jar Test: this test is used to assess the supernatant quality after settling for a fixed time. It may also give an idea about the type of floc. In addition, the test is used to determine the optimum pH and optimum coagulant dose according to supernatant quality tests.

(2) The second type of jar test is used to determine the feasibility of dissolved air flotation for floc removal.

(3) Settling Velocity Measurement: in this test, samples are taken at time intervals at a specified depth from the surface of the water in the beaker.

Hudson and Wagner (1980) stated that the sedimentation jar test (type 1) may be misleading and it is better to use type (3) for determining optimum pH and dose.

The type (1) test considers the flocs that settle fast and does not account for flocs that posses lower settling velocity.

The jar test that measures velocity distribution is more important when the effect of coagulant aids is to be assessed. Since the effect of these aids is to improve floc characteristics and mainly settleability and filterability, this can only be done if velocity distribution is compared.

The effect of sedimentation time can only be assessed if the type (3) test is used.

For the purpose of this research, the type (3) jar test was chosen because it was intended to determine the effect of sedimentation time on removal of algae. Also, because it was desired to evaluate the effect of coagulant aids on the coagulation-sedimentation process.

4.2.2 Assessment of Solid-Liquid Separation in the Jar Test

There are some parameters that should be determined during all steps of the test. For example, the mean velocity gradient G, and the time for both flash mix and flocculation should be determined. These can be varied to find the optimum or set at predetermined levels from the beginning of the experiments.

The main objective of the coagulation process is the effectiveness of separating solids from the liquid. So, it is important to decide which parameters are most appropriate to assess such efficiency for an existing or proposed method of solid-liquid separation. A variety of parameters and techniques were applied. Tekippe and Ham (1970, 1970a) and Bratby (1980) listed twenty two criteria to judge the performance of the coagulation process and discussed the merits of some of the techniques.

In the sedimentation jar test type (1), settleability properties and time of coagulation have been used to evaluate the optimal pH and coagulant dose (Eckenfelder and Ford, 1970).

A filterability number, F, was defined by Ives (1986) to evaluate the suitability of a certain media for filtration. This technique was considered as useful and may be used together with the jar test (Hilson and Richards, 1980).

Camp (1968) used floc volume concentration and floc size distribution to evaluate the effect of mixing on the coagulation process. Floc strength and size may also be a measure of settling properties and velocity distribution (Tambo and Watanabe, 1979).

Tekippe and Ham (1970a) divided these tests into three categories:

(1) Techniques based upon particle charge, such as colloid titration, cation exchange capacity (CEC), zeta potential...etc. These techniques are not useful for finding optimum dose, although useful when initial charge is high.
(2) Those which measure speed of floc formation, size count and settleability. Such methods are useful when settling tanks are the means of solid-liquid separation.
(3) Filtration-related techniques. These are useful to evaluate filtration properties of a suspension but do not give information about settleability.

Since the whole theme of the coagulation process is to optimise the coagulant dose and pH in terms of the performance of the solids liquid-separation process, it is reasonable to assume that tests which measure settleability and filterability are most suitable if sedimentation and filtration processes are to follow.

Packham (1972) reported that measurement of refiltration rate gave very poor results when applied to alum floc at the Water Research Association (UK). Also, measurement of floc-related properties such as size and shear strength, although indirectly related to settleability, brought difficulty and very poor reproducibility of results.

Settleability tests were reported to predict closely the optimum dosage required for filtration (Adin and Rebhun, 1974). Habibian and O'Melia (1975) found that settling jar test results correlated well with the performance of a pilot plant filter.

Benedeck and Bancsi (1976) reported that multiple sampling during quiescent settling (jar test type 3) has been shown to be a valid technique to predict the performance of a full scale clarifier.

For the purpose of this research, measurement of turbidity for multiple samples was chosen. Thus velocity distribution can be constructed for the forming flocs. Such tests may reflect settleability and filterability of the suspension treated.

4.2.3 Coagulants Tested

Three coagulants were used in this work: alum, kaolinite and bentonite. The alum used has a formula of $Al_2SO_4.14H_2O$ supplied as the equivalent of 17% AL_2O_3 with 0.01% Fe and an insoluble residual of 0.12%.

Kaolinite was obtained from Mahis mines in Jordan. Its chemical analysis was given by Abed (1982) and is presented here in Table (4.2). Bentonite ores seem to be not available in Jordan although some clays were found containing smectites in small proportions (ibid). Surrey powder, obtained from Laporte Earths (Surrey,UK), was the bentonite used in this work. The chemical analysis was given by the supplier mentioned above and is shown in Table (4.2)

4.2.4 Jar Test Equipment

Jar tests in this work were carried out using Phips and Bird equipment (Richmond, Va., U.S.A.). The device consists of six stirrers each ending with a blade 7.62 cm in diameter and 2.54 cm height. The bottom edge of the blade is 5.72 cm above the beaker's bottom. Six beakers with 2 l samples were used to carry out the test.

A peristaltic pump was used to take samples at 5 cm depth below the water surface at specified time intervals from the six beakers, simultaneously.

4.2.5 Procedure

Grab samples were collected from the effluent of Al-Samra wastewater stabilization ponds in Jordan. Jar testing was carried out immediately on collected samples.

The procedure followed to carry out the jar test of type (3) (see Section 4.2.1) incorporating multiple sampling during quiescent settling is based on the method adopted by the British Water Research Association. The method was explained by Packham (1972), Benedek and Bancsi (1976) and Hudson and Wagner (1980) with some variations among these investigators. The sequence of

Table 4.2: Chemical analysis for kaolinite and bentonite used in this work

component	kaolinite (%)	bentonite (%)
Sio_2	60.7	55.2
Al_2O_3	25.6	13.7
Fe_2O_3	1.78	8.1
FeO	< 0.1	-
TiO_2	1.44	0.7
MnO	< 0.02	-
CaO	~ 0.1	6.3
MgO	0.28	3.3
Na_2O	0.24	trace
K_2O	0.78	0.6
P_2O_5	0.07	-
H_2O^-	0.66	-
H_2O^+	8.24	-

the procedure was taken from Eckenfelder and Ford (1970) and Eckenfelder (1989). The procedure followed is summarized below:

(1) An approximate dose of the coagulant was determined by stirring a sample of 200 ml and addition of increments of coagulant solution until first floc is formed.

(2) Turbidity of the raw sample was measured.

(3) Six 2 l samples were placed in the beakers and pH in the beakers was adjusted in six increments.

(4) The samples were mixed at 120 rpm for 1 minute, the approximate coagulant dose was added and stirring continued at the same speed for another one minute.

(5) pH was measured for the six beakers after addition of the coagulant.

(6) The speed was reduced to 40 rpm for the flocculation step, which lasted 15 minutes.

(7) The stirrer was stopped, the flocs were allowed to settle and samples were taken at 5 cm below the water surface for analysis. This determines the optimum pH.

(8) Using the optimum pH determined above, steps 1-to-7 were repeated employing different coagulant doses. Doses were increased by a factor of $\sqrt{2}$ (Lockyear and Stevenson, 1986).

(9) Samples were taken at zero time and after 1, 2, 3, 6, 9, 40 minutes, 60 minutes, 2 hours,1 day, 2 days, and 3 days from the beginning of the settling period.

(10) All samples were analyzed for turbidity. Samples after 40 minutes and 3 days were analyzed also for TSS, VSS, chlorophyll-a, total and soluble COD, total and soluble BOD, NH_3, TKN, TP and PO_4^{-3}.

(11) When coagulant aid was used it was added 1/2 minute after addition of the primary coagulant.

Expressing turbulence intensity in terms of rpm does not help in comparing between different work because the mean velocity gradient, G,

depends on power input, shape and dimensions of the jar (Lai et al., 1975). Many methods have been used to calibrate jar test equipment for the relation between rpm and G (see e.g. Ogedengbe, 1976; Leentvaar and Ywema, 1980; Mhaisalkar et al., 1986).

The results of the studies carried out by Camp (1968) and Camp and Conklin (1970) were used to calculate the G value employed in this work. The volume of the sample in this work is similar to that in the above mentioned studies. Also, the dimensions of the jars are nearly alike. Accordingly, the flash mix intensity employed in the experiments of this work was 80 s^{-1} (120 rpm), and that of the flocculation step was 15 s^{-1} (40 rpm).

4.3 Crossflow Microfiltration

A laboratory scale facultative pond 1m x 1m x 0.30m was set up in the laboratory and was illuminated by fluorescent lamps. The effluent of the pond was collected and used as a feed for a crossflow microfiltration apparatus. The membrane used was of the woven cloth type with 98.0 cm length and 3.1 cm circumference. Tests were carried out at a pressure drop of 0.6 bar and the concentrate was discarded and not recycled into the feed tank. The feed tank was filled manually with 25 l of the facultative pond effluent, and was refilled when the level of water dropped below the 25 l mark.

The crossflow filtration process was carried out without laying down a dynamic membrane on the cloth using alum or any other coagulant.

Four runs were carried out and each run continued for 4 hours. Samples were taken at time intervals, from both the feed and permeate, and were analyzed for turbidity. The flow rate of the permeate was also measured at these time intervals.

In the third run, an alum dose of 10 mg/l was added to the feed and TSS concentrations were measured, in addition to turbidity, in the collected samples.

4.4 Methods of Measurement and Examination

Methods of measurement for different parameters are listed in Table (4-3). All samples were analyzed according to the Standard Methods for the Examination of Water and Wastewater (APHA et al., 1985).

Table 4.3: Methods of measurement of different parameters

ABBREVIATION	DESCRIPTION	METHOD
TSS	total suspended solids	209-C
VSS	volatile suspended soli	209-D
TUR	turbidity	nephelometric
CHL	chlorophyll - a	1002G fluorometric
BOD_t	total 5-day biochemical oxygen demand	507 bottle test. for DO 421-B
BOD_S	soluble 5-day biochemical oxygen demand	507 bottle test for Do 421-B
COD_t	total chemical oxygen demand	508-B closed reflux titrimetric
COD_S	soluble chemical oxygen demand	508-B closed reflux titrimetric
PO_4^{-3}	orthophosphate	424-E stanous chloride
TP	total phosphorus	424-C,II sulphuric acid-nitric acid
NH_3	ammonia	417-B Nessler
TKN	total Kjeldahl nitrogen	420-A macro
T	temperature	thermometer
pH	hydrogen ion concentration	electrode
	algal species	microscopy
Q	flow	time-volume

CHAPTER 5

Results

5.1 Rock Filter Influent Characteristics

All parameters that were measured for the influent to the rock filters are presented in Table (A-1) in Appendix A.

In Chapter 3 it was pointed out that influent TSS concentration affects the process of removal of TSS in rock filters. Table (5.1) shows the monthly average of some parameters, including TSS in the influent to rock filters. VSS and chlorophyll-a averages are also presented in this table.

5.1.1 Variation of TSS in the Influent

In analyzing the data, care should be taken not to confound the effects of experimental uncertainty in the measurements of certain parameters, with those of a sloping response surface due to change in the levels of an influent variable. To analyze for a specific response, the input factors must be set at fixed levels.

In this work, hydraulic loading, diameter of the rock and depth of sampling points were all controllable factors and were set at the required levels mentioned in the experimental design.

On the other hand, influent total suspended solids concentration changed according to changes in the wastewater stabilization ponds system from which the influent to the rock filters was taken. Average values should be taken to represent the levels of TSS or chlorophyll-a in the influent for each filter run.

Fig. (5.1) is plotted using the data of Table (5.1). It shows the monthly average variation in concentration of TSS, VSS and chlorophyll-a in the influent throughout the test period.

Tble 5.1: Monthly averages of some influent characteristics

month	TSS (mg/l)	VSS (mg/l)	turbidity (NTU)	chlorophyll (µg/l)
DEC/90	255 (6) *	238 (10)	219 (9)	390 (24) *
JAN/91	225 (30)	216 (29)	205 (38)	326 (33)
FEB/91	142 (24)	130 (27)	121 (21)	203 (52)
MAR/91	158 (35)	140 (31)	126 (30)	221 (48)
APR/91	194 (21)	166 (20)	153 (14)	263 (34)
MAY/91	207 (68)	167 (53)	173 (50)	238 (107)
JUN/91	178 (32)	140 (27)	160 (30)	178 (74)
JUL/91	212 (60)	170 (53)	177 (53)	393 (47)
AUG/91	290 (80)	232 (64)	243 (59)	407 (38)

* values between brackets are standard deviations

Table 5.2: Some influent parameters averaged over a period one week before and one week after each run.

experi–mental set	hydraulic loading (m3/m3.d)	TSS (mg/l)	VSS (mg/l)	turbidity (NTU)	chlorophyll (µg/l)
1	0.1	248 (17)*	232 (15)	215 (13)	382 (27)*
1	0.5	154 (40)	143 (42)	133 (37)	221 (66)
1	1	138 (29)	125 (28)	114 (21)	197 (54)
1	1.5	227 (31)	217 (29)	205 (38)	332 (35)
1	2	211 (40)	203 (39)	190 (48)	304 (45)
2	0.1	159 (39)	145 (36)	123 (31)	224 (59)
2	0.5	183 (33)	158 (31)	142 (21)	252 (39)
2	1	152 (26)	138 (20)	122 (21)	225 (26)
2	1.5	188 (27)	159 (23)	144 (13)	255 (36)
2	2	159 (36)	142 (34)	128 (31)	220 (54)
3	0.1	184 (34)	145 (25)	157 (30)	363 (75)
3	0.5	192 (68)	152 (53)	161 (51)	180 (93)
3	1	217 (65)	178 (50)	180 (47)	270 (97)
3	1.5	255 (66)	202 (53)	213 (55)	403 (33)
3	2	178 (29)	142 (25)	160 (28)	214 (107)
3	1 repetition	293 (88)	237 (71)	245 (65)	416 (34)
2	1 repetition	192 (12)	165 (15)	165 (8)	256 (31)

* values between brackets are standard deviations

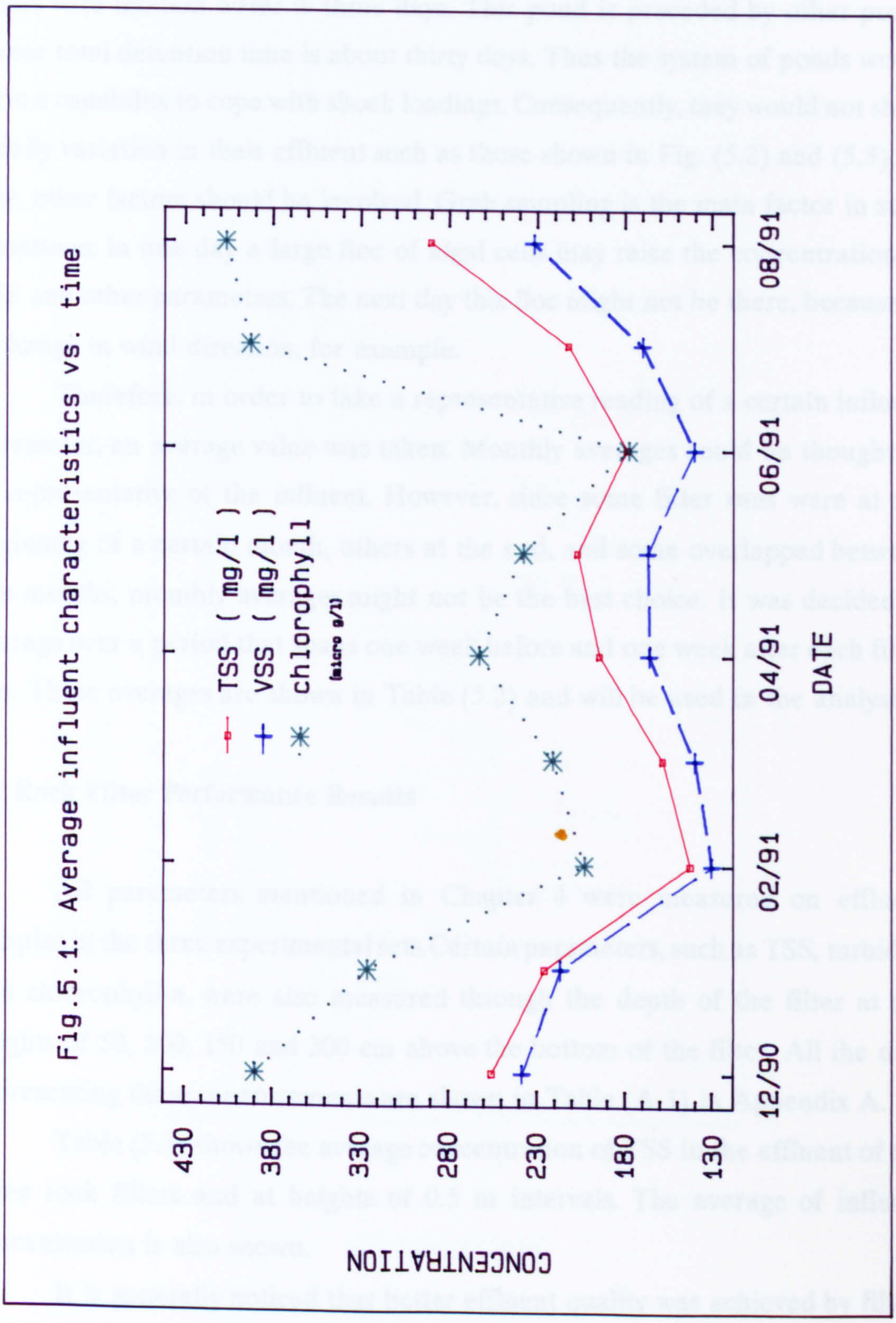


Fig.5.1: Average influent characteristics vs. time

Fig. (5.2) and (5.3) depict the daily variation of these influent parameters. However, these daily variations should not be attributed to changes in the system of wastewater stabilization ponds, which have long detention time and is not susceptible to shock loadings. The detention time of the pond supplying the pilot filters with influent water is three days. This pond is preceded by other ponds whose total detention time is about thirty days. Thus the system of ponds would have a capability to cope with shock loadings. Consequently, they would not show a daily variation in their effluent such as those shown in Fig. (5.2) and (5.3). In fact, other factors should be involved. Grab sampling is the main factor in such variations; in one day a large floc of algal cells may raise the concentration of TSS and other parameters. The next day this floc might not be there, because of a change in wind direction, for example.

Therefore, in order to take a representative reading of a certain influent parameter, an average value was taken. Monthly averages could be thought of as representative of the influent. However, since some filter runs were at the beginning of a certain month, others at the end, and some overlapped between two months, monthly averages might not be the best choice. It was decided to average over a period that spans one week before and one week after each filter run. These averages are shown in Table (5.2) and will be used in the analysis.

5.2 Rock Filter Performance Results

All parameters mentioned in Chapter 4 were measured on effluent samples in the three experimental sets.Certain parameters, such as TSS, turbidity and chlorophyll-a, were also measured through the depth of the filter at the heights of 50, 100, 150 and 200 cm above the bottom of the filter. All the data representing these measurements are shown in Table (A-1) in Appendix A.

Table (5.3) shows the average concentration of TSS in the effluent of the three rock filters and at heights of 0.5 m intervals. The average of influent concentration is also shown.

It is generally noticed that better effluent quality was achieved by filters containing smaller rock size. The results presented in Table (5.3) reveal that

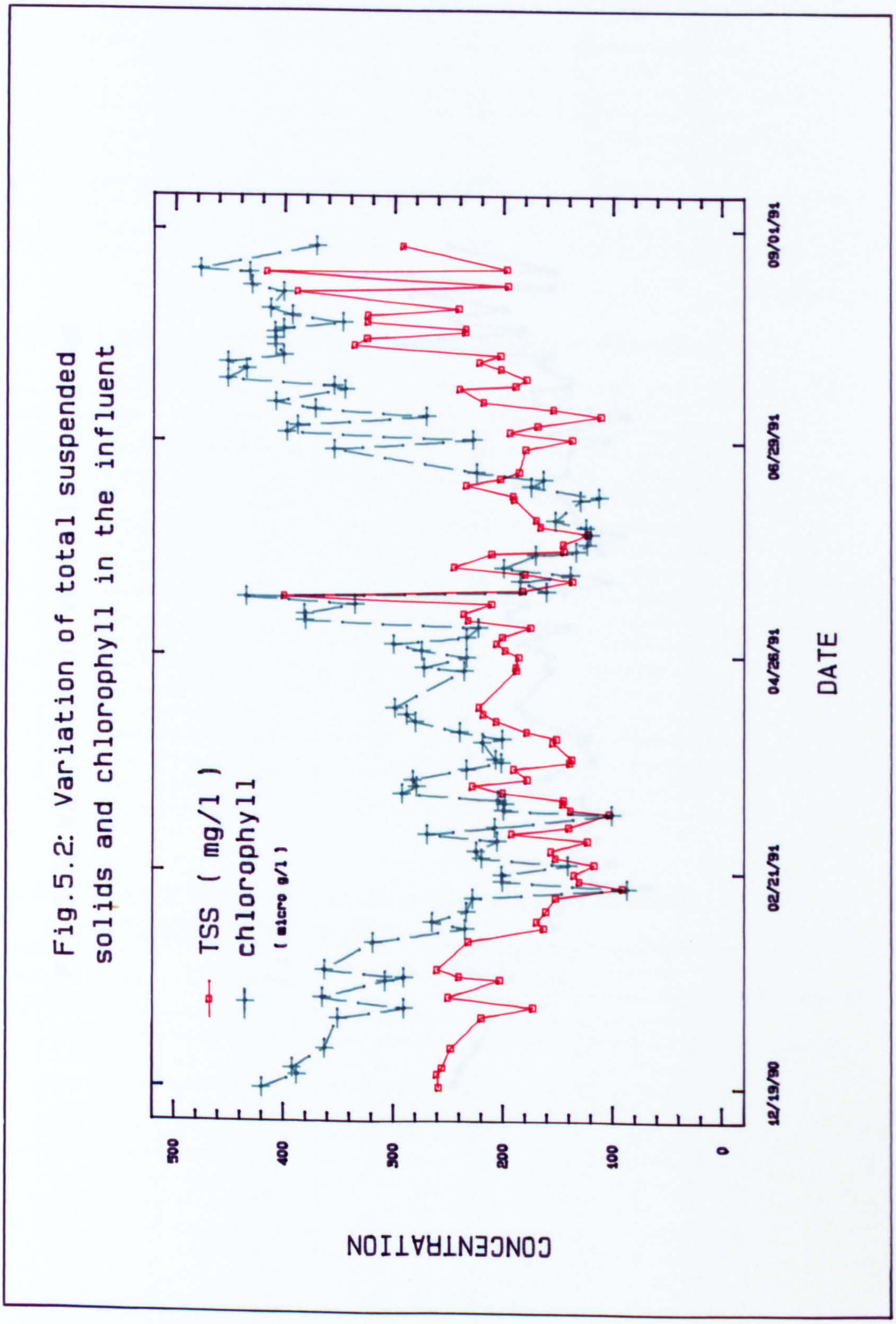


Fig.5.2: Variation of total suspended solids and chlorophyll in the influent
TSS (mg/l)
chlorophyll (micro g/l)
500
400
300
200
100
0
12/19/90
02/21/91
04/26/91
06/29/91
09/01/91
DATE
CONCENTRATION

Fig.5.3: Variation of volatile suspended solids and chlorophyll in the influent

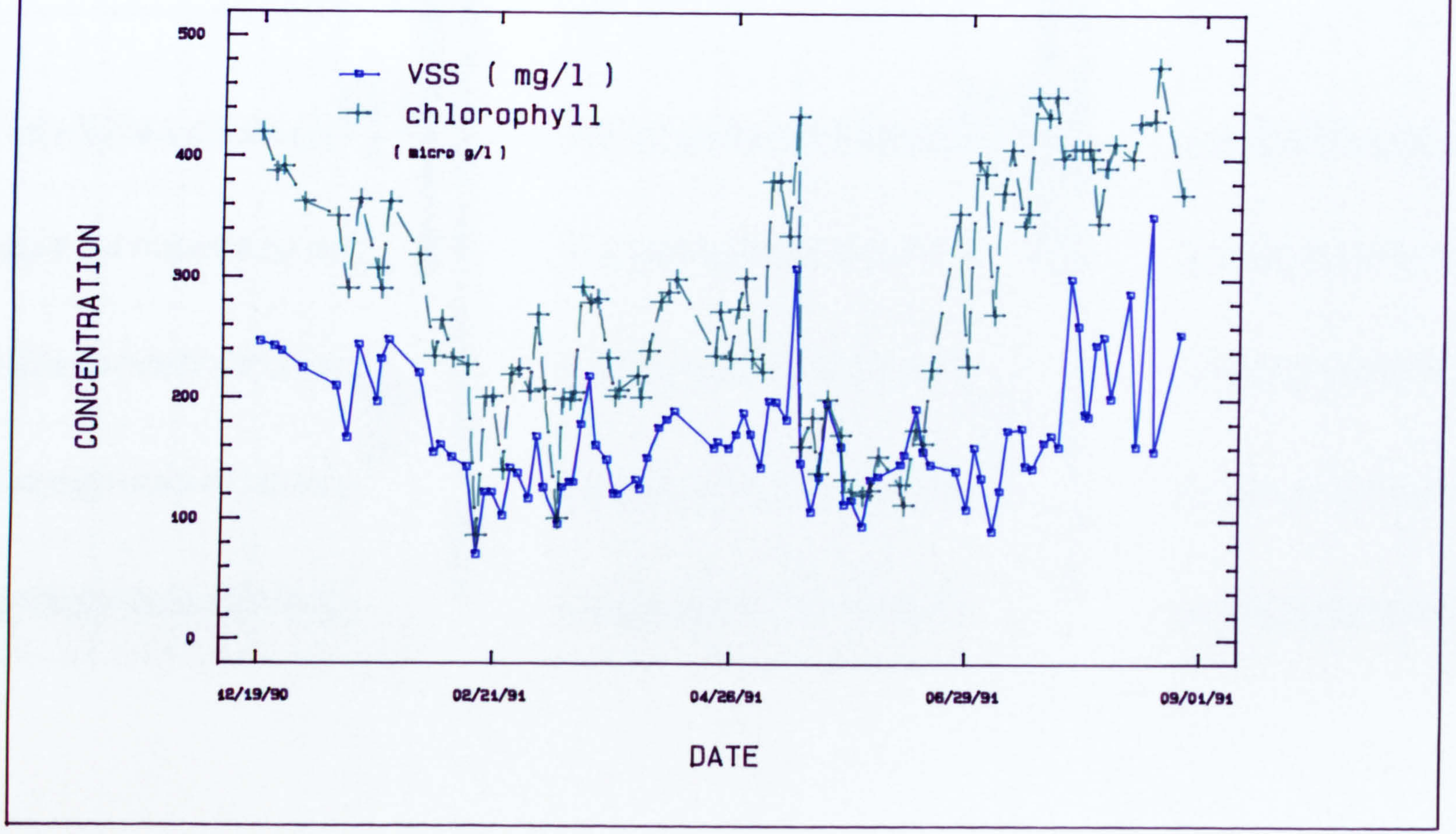


Table 5.3 : Suspended solids concentration at different heights of the rock filter

hydraulic loading (m3/m3.d)	set No.	TSSin aaverage (mg/l)	rock 10 cm			
			height (m)			
			0.5	1.0	1.5	2.0
0.1	1	248	76	75	74	73
0.1	2	159	74	73	72	72
0.1	3	184	80	78	77	75
0.5	1	154	80	78	78	77
0.5	2	183	90	89	88	88
0.5	3	192	91	89	88	88
1.0	1	138	87	86	86	86
1.0	2	152	89	88	88	88
1.0	3	217	112	108	108	108
1.5	1	227	148	147	145	145
1.5	2	188	133	132	132	132
1.5	3	255	159	156	154	153
2.0	1	211	168	163	161	158
2.0	2	159	118	118	117	118
2.0	3	178	147	141	136	132

Table 5.3 contd: Suspended solids concentration at different heights of the rock filter

hydraulic loading (m3/m3.d)	set No.	TSSin aaverage (mg/l)	rock 5 cm			
			height (m)			
			0.5	1.0	1.5	2.0
0.1	1	248	64	63	62	62
0.1	2	159	63	61	60	59
0.1	3	184	73	66	64	63
0.5	1	154	74	72	70	69
0.5	2	183	80	78	78	77
0.5	3	192	82	80	79	77
1.0	1	138	82	81	80	79
1.0	2	152	82	81	81	80
1.0	3	217	103	99	97	96
1.5	1	227	143	140	138	136
1.5	2	188	120	119	119	119
1.5	3	255	152	150	149	148
2.0	1	211	150	146	145	143
2.0	2	159	107	106	106	105
2.0	3	178	141	127	124	119

Table 5.3 contd: Suspended solids concentration at different heights of the rock filter

hydraulic loading (m3/m3.d)	set No.	TSSin aaverage (mg/l)	rock 1 cm			
			height (m)			
			0.5	1.0	1.5	2.0
0.1	1	248	43	41	40	39
0.1	2	159	44	38	35	33
0.1	3	184	42	39	38	37
0.5	1	154	68	63	58	54
0.5	2	183	62	60	59	58
0.5	3	192	70	64	60	58
1.0	1	138	70	68	65	62
1.0	2	152	61	59	60	59
1.0	3	217	75	72	71	70
1.5	1	227	110	105	103	100
1.5	2	188	90	88	86	85
1.5	3	255	109	102	99	97
2.0	1	211	134	127	121	116
2.0	2	159	81	80	78	77
2.0	3	178	95	91	90	89

most of the TSS removal occurred in the first 50 cm of the filter height. The rest of the filter height made less contribution to the treatment process.

Influent average concentrations and effluent average concentrations were used to produce Fig. (5.4), (5.5) and (5.6). In these figures, percentage removal for both TSS and chlorophyll-a is plotted against hydraulic loading for the three filters.The plots indicate that rock filters accomplished 20% to 85% removal of TSS, depending on rock size and hydraulic loading. Higher percentages of removal were obtained in case of chlorophyll-a by comparison with TSS.

Average values of both soluble and total COD and BOD_5 for influent and effluent of the filters are presented in Table (5.4). It is observed that a drop in total COD and total BOD has taken place in the three filters. For soluble COD and BOD, a decrease in concentration was noticed most of the time.

Table (5.5) introduces average concentrations of nutrients in influent and effluent of the rock filters.

Removal of ammonia was effected by the filters most of the time. However, occasional increase in ammonia concentration was noticed in the effluent over the influent of the filters. The increase in TKN was less frequent.

Total and soluble phosphorus were often reduced by the filters but an increase was noticed sometimes.

5.3 Jar Test Results

The jar tests were carried out to determine optimum pH and optimum coagulant dose for each of the coagulants tested. Also, the tests were carried out with the aim of determining the effect of settling time on the process of coagulation. The following sections offer a brief presentation of the results of the jar tests.

Fif.5.4: Suspended solids and

chlorophyll removal for rock 1cm

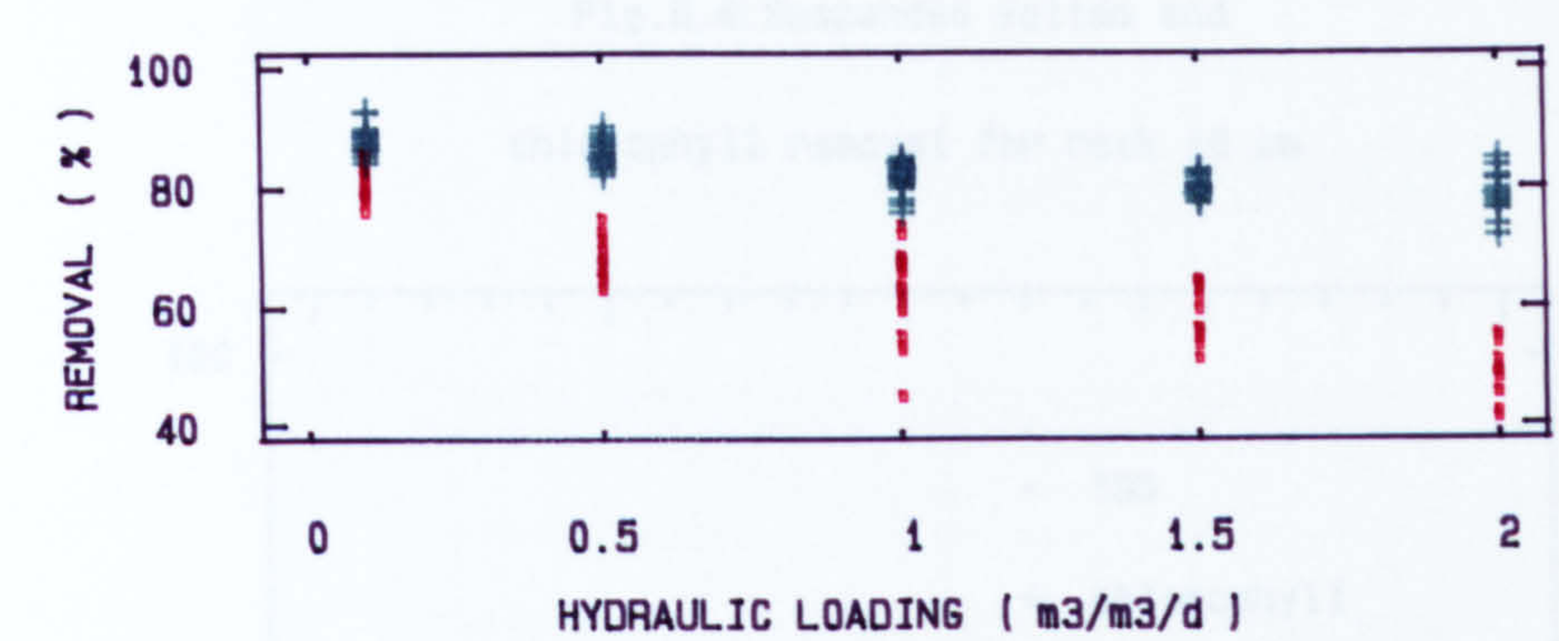


Fig.5.5: Suspended solids and

chlorophyll removal for rock 5 cm

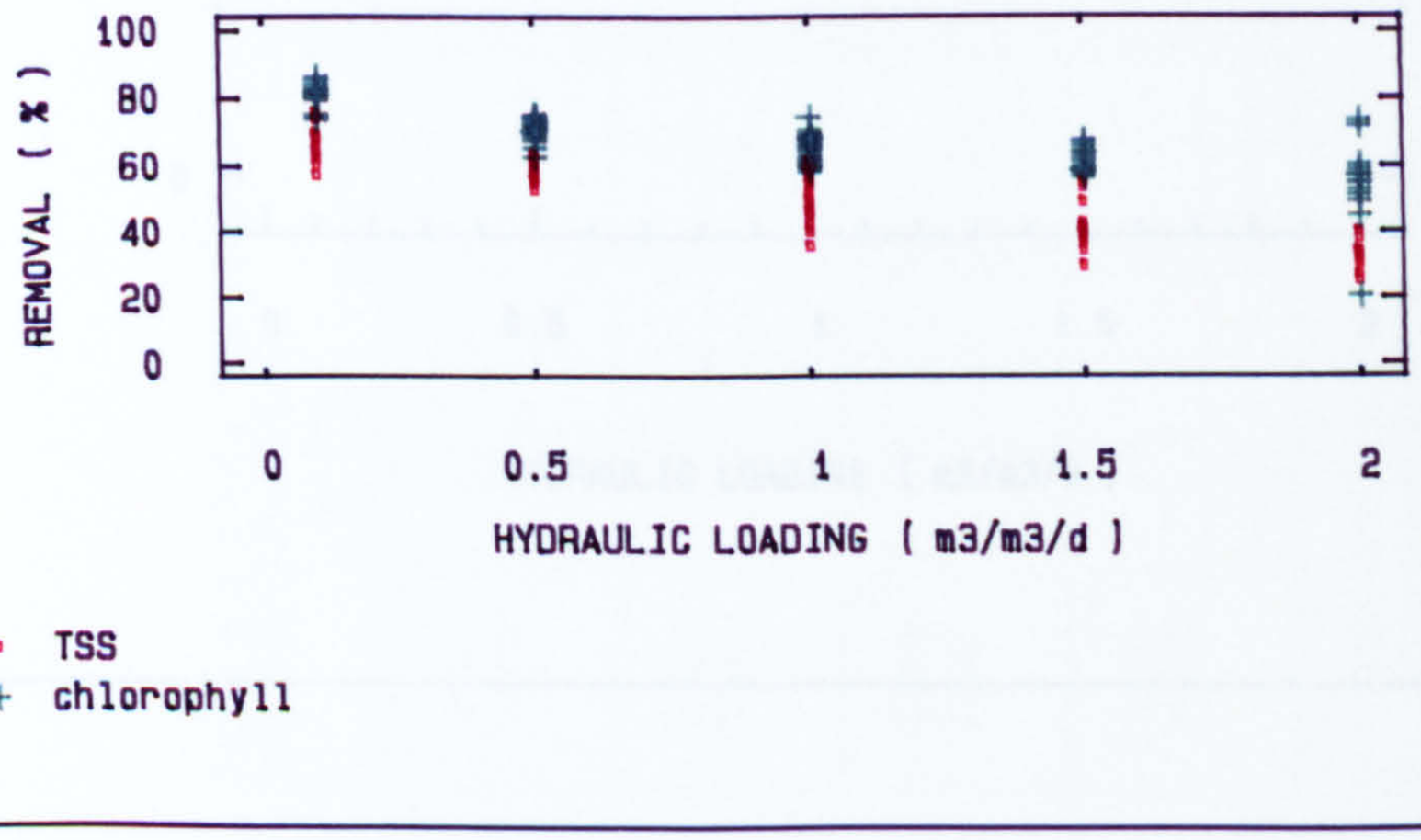


Fig.5.6: Suspended solids and chlorophyll removal for rock 10 cm

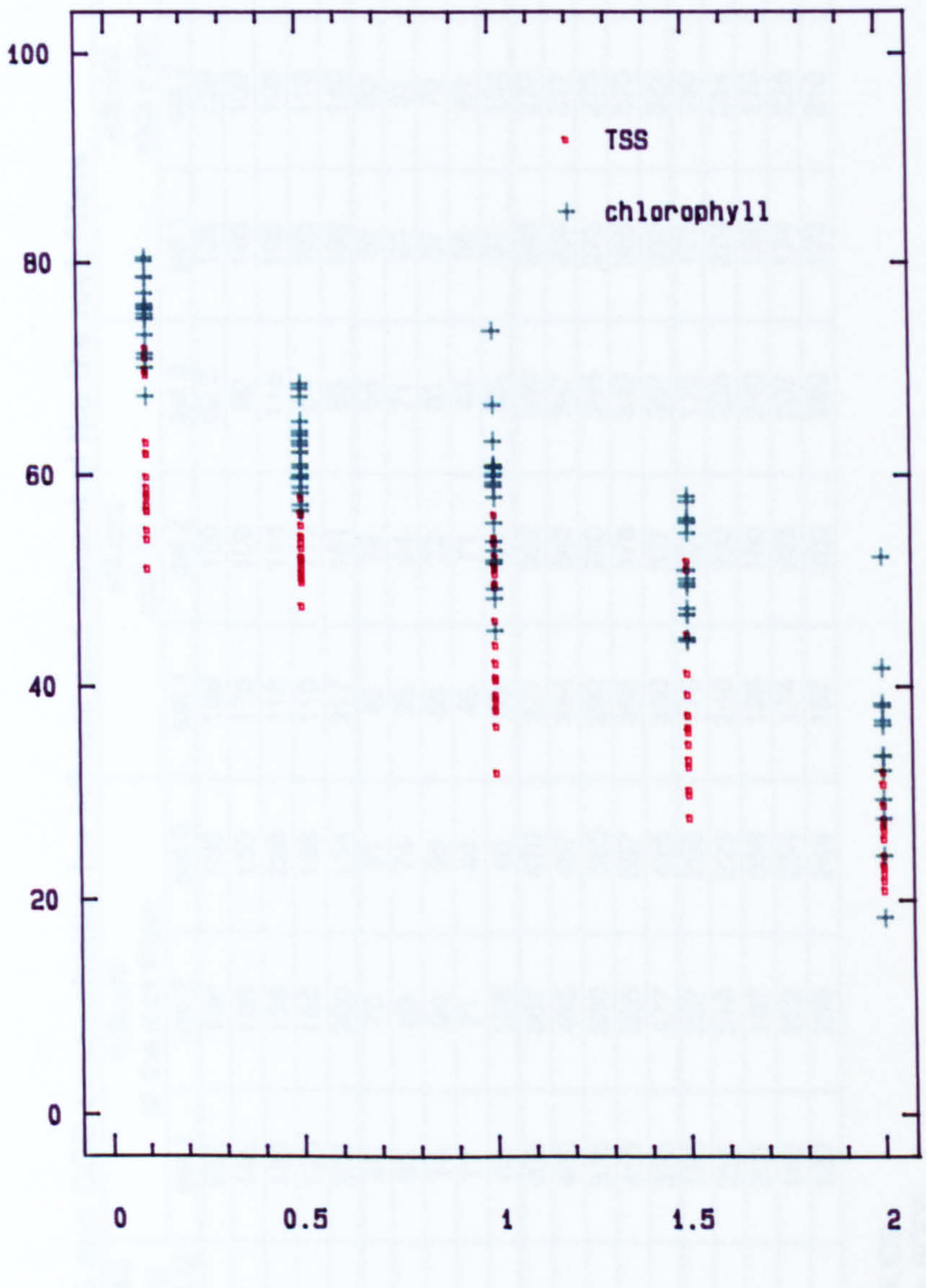


Table 5.4: BOD5 and COD concentration for influent and effluent of the the rock filters

parameter	hydraulic loading (m3/m3.d)	influent all the rock filters			effluent rock 10 cm			effluent rock 5 cm			effluent rock 1 cm		
		set 1	set 2	set 3	set 1	set 2	set 3	set 1	set 2	set 3	set 1	set 2	set 3
* BODt (mg/l)	0.1	250	138	135	126	120	62.3	106	128	50	80	111	53
	0.5	195	138	150	176	108	90	188	102	94	165	79	142
	1.0	136	168	228	118	114	116	106	105	112	94	93	104
	1.5	199	168	188	110	115	84	125	110	84	122	83	60
	2.0	235	200	154	217	161	85	193	140	80	170	130	68
** BODs (mg/l)	0.1	45	70	51	40	72	54	38	92	40	38	92	47
	0.5	95	69	74	95	64	71	95	51	52	95	54	57
	1.0	70	96	96	68	72	81	57	70	73	60	66	70
	1.5	77	71	44	49	71	43	56	64	46	64	64	31
	2.0	60	128	68	58	80	55	57	104	52	58	58	41
+ CODt (mg/l)	0.1	400	382	470	271	302	240	248	272	191	202	222	181
	0.5	400	495	457	344	342	347	334	317	307	300	302	250
	1.0	318	380	685	292	280	345	275	275	320	258	230	300
	1.5	418	526	597	362	376	346	365	370	273	338	346	220
	2.0	370	477	425	330	337	387	335	322	350	320	317	265
++ CODs (mg/l)	0.1	180	257	233	175	203	175	175	190	149	175	170	146
	0.5	225	318	223	240	232	228	229	214	200	227	204	183
	1.0	205	187	368	198	180	240	195	170	226	192	156	216
	1.5	182	252	254	174	225	207	174	228	168	171	205	144
	2.0	180	253	249	167	220	280	173	210	225	153	206	188

* BODt = total BOD5
** BODs = soluble BOD5
\+ CODt = total COD
++ CODs = soluble COD

Table 5.5: Nutrients concentration in the influent and effluent of the the rock filters

parameter	hydraulic loading (m3/m3.d)	influent all the rock filters			effluent rock 10 cm			effluent rock 5 cm			effluent rock 1 cm		
		set 1	set 2	set 3	set 1	set 2	set 3	set 1	set 2	set 3	set 1	set 2	set 3
NH3 (mg/l)	0.1	3.5	9.7	2.3	3.3	11.2	0.8	3.4	10.5	0.7	3.2	10.4	0.6
	0.5	11.0	6.3	0.3	11.0	9.4	1.4	11.0	5.8	1.7	11.0	6.8	2.3
	1.0	4.0	8.0	0.2	3.8	7.8	0.1	1.8	7.9	0.1	2.5	7.6	0.1
	1.5	4.0	10.9	5.0	4.0	10.9	3.8	4.0	11.0	4.7	4.0	11.1	4.2
	2.0	9.2	10.9	0.4	9.8	11.1	0.4	9.2	11.1	0.4	8.5	11.1	0.3
TKN (mg/l)	0.1	20.0	20.4	15.9	16	19.9	14.7	15	18.7	14.4	12.3	19.1	12.2
	0.5	20.0	15.0	14.6	19.8	19.6	16.2	19.7	17.9	16.3	18	16.5	16.0
	1.0	12.6	20.1	16.9	9.8	19.4	17.6	9.7	17.3	15.8	8.5	15.7	15.6
	1.5	12.0	22	16.5	11.5	21.5	16.4	10.0	21.5	15.1	9.7	18.0	14.8
	2.0	22.2	19.6	15.1	22.0	19.1	13.8	21.0	18.7	31.2	17.0	17.9	13.0
soluble P (mg/l)	0.1	6.0	6.0	3.7	5.9	6.0	4.9	6.0	6.0	4.6	5.9	5.6	2.3
	0.5	5.1	4.4	2.0	5.1	5.8	3.3	5.1	5.9	3.4	5.0	5.2	2.6
	1.0	5.9	4.8	4.7	5.9	4.7	4.6	5.9	4.7	4.0	5.7	4.0	3.6
	1.5	5.7	4.5	3.4	5.6	5.4	3.3	5.6	5.3	2.7	5.6	5.3	1.4
	2.0	5.0	3.7	3.2	5.0	3.3	2.6	5.0	2.9	3.9	5.0	2.3	2.6
total P (mg/l)	0.1	6.5	6.3	5.2	6.4	6.0	5.4	6.3	6.0	5.3	6.0	5.7	3.2
	0.5	5.6	5.4	5.3	5.5	6.6	4.9	5.4	6.4	5.0	5.2	5.4	4.2
	1.0	6.0	6.5	8.6	5.9	6.1	7.8	5.9	6.0	7.4	5.7	3.2	7.0
	1.5	6.0	5.0	5.1	5.8	5.8	4.4	5.8	5.7	4.7	5.7	5.4	3.8
	2.0	5.5	4.1	5.7	5.4	3.7	4.7	5.4	3.4	4.5	5.3	3.0	4.2

5.3.1 Optimum pH

The first part of the jar test was carried out to determine the optimum pH. Fig. (5.7), (5.8) and (5.9) show the variation of turbidity remaining versus pH of the dispersion.

The optimum pH for alum coagulation was found to be about 5.3. For kaolinite the optimum pH occurred at a value of 9.0 when supernatant turbidity was measured after 40 minutes. However, Fig. (5.8b) shows that the optimum pH shifted towards neutral, between 7-8, when supernatant turbidity was measured after 1 day and 3 days. The same trend of lowering optimum pH for longer settling time was noticed for bentonite. Optimum pH was found to be 9 for 40 minutes readings, and optimum pH was 7.3 for readings after one day and three days settling.

5.3.2 Optimum Coagulant Dose

Fig. (5.10) shows that minimum remaining concentration of TSS after settling for 40 minutes occurred at 460 mg/l of alum. Further increase of alum dose deteriorated the performance. Then improvement took place again with more increase of alum dose.

The fraction of turbidity remaining, which is the turbidity at any time divided by turbidity at the start of settling period (t=0), is plotted against time in Fig. (5.11) for small doses of alum. It can be noticed that removal of flocs increased with sedimentation time at low alum doses. At alum doses above 80 mg/l, negligible floc settling was achieved with increase of sedimentation time beyond 40 minutes. Also, the figures reveal that better settling flocs accompanied increase in alum dose.

Increase in kaolinite dose from 707 mg/l to 8000 mg/l resulted in a parallel improvement in removal of flocs (Fig. 5.12a, 5.12b). These Figures also show that the fraction of turbidity remaining decreased with settling time, especially at lower doses of kaolinite.

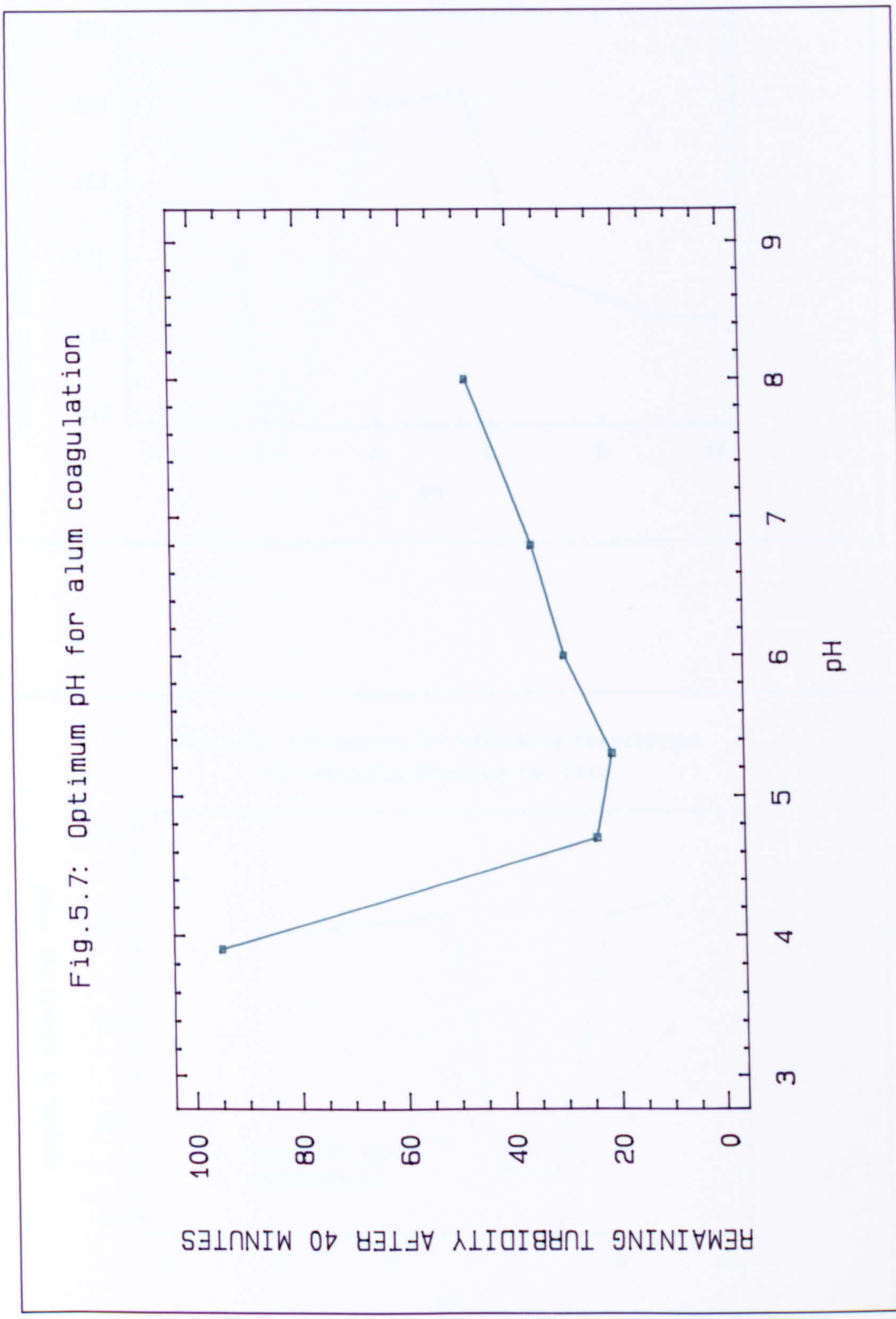


Fig.5.7: Optimum pH for alum coagulation

Fig.5.8a: Optimum pH for kaolinite coagulation
for 40 minutes settling jar test

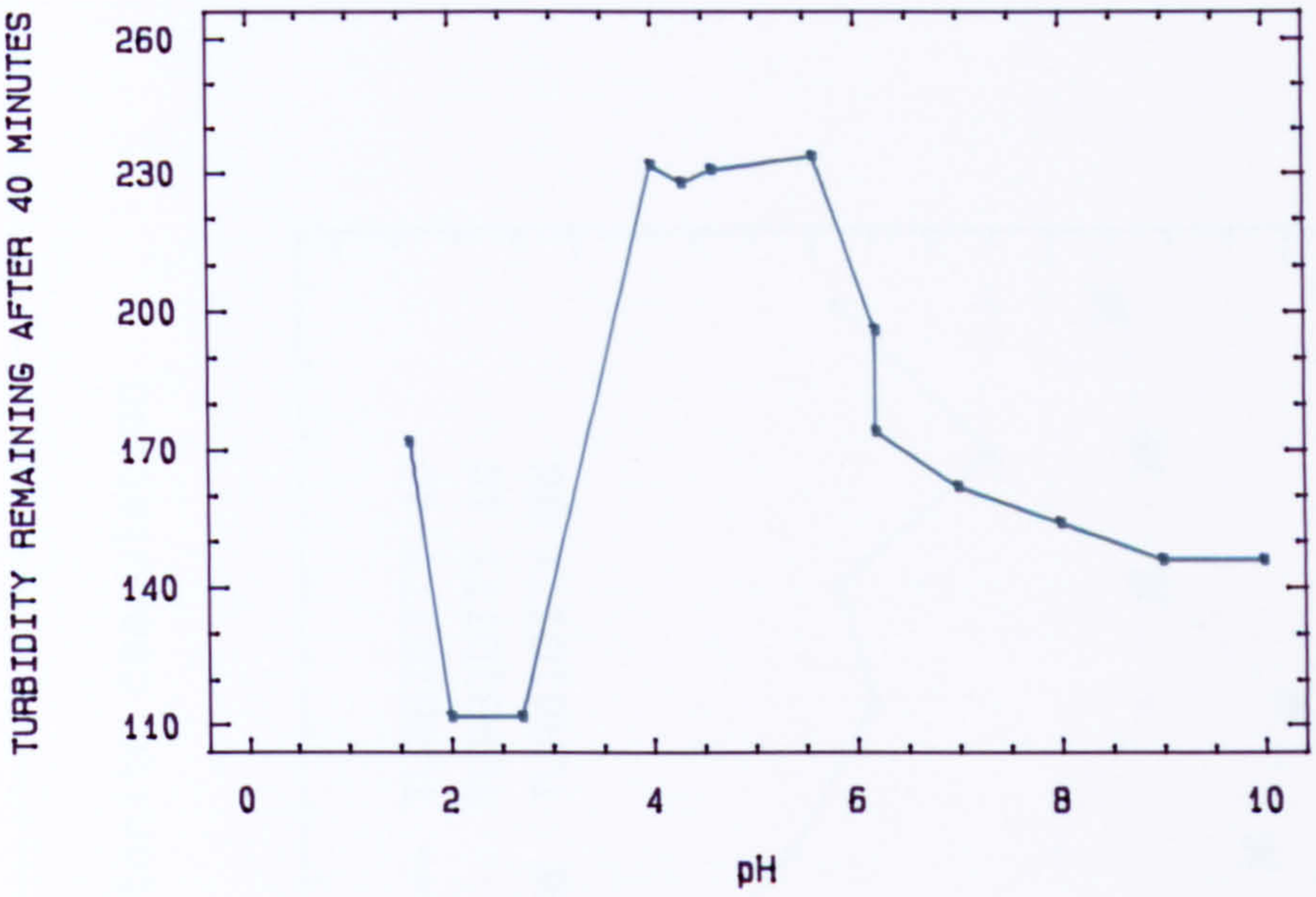
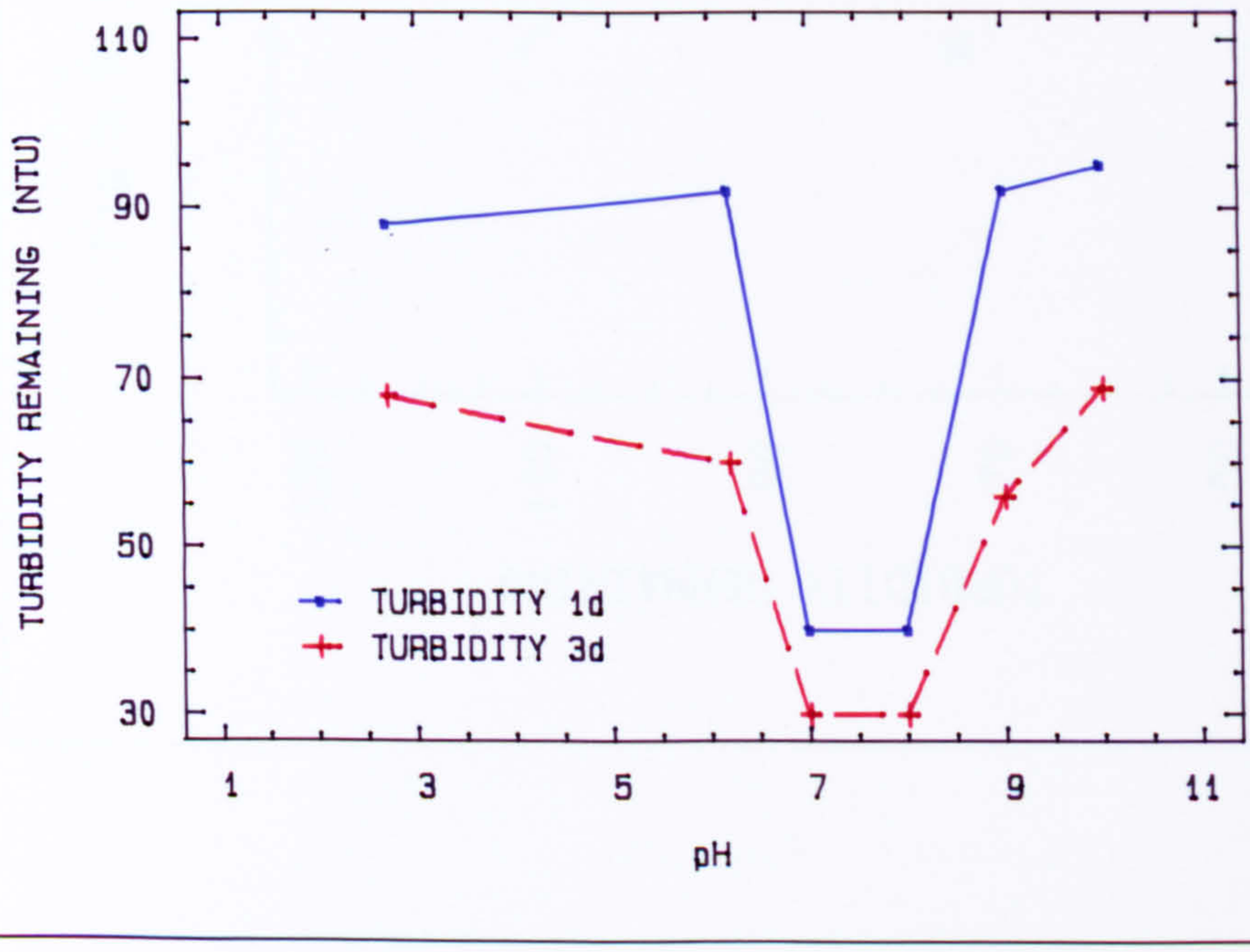


Fig.5.8b: Optimum pH for kaolinite coagulation
for extended settling jar test



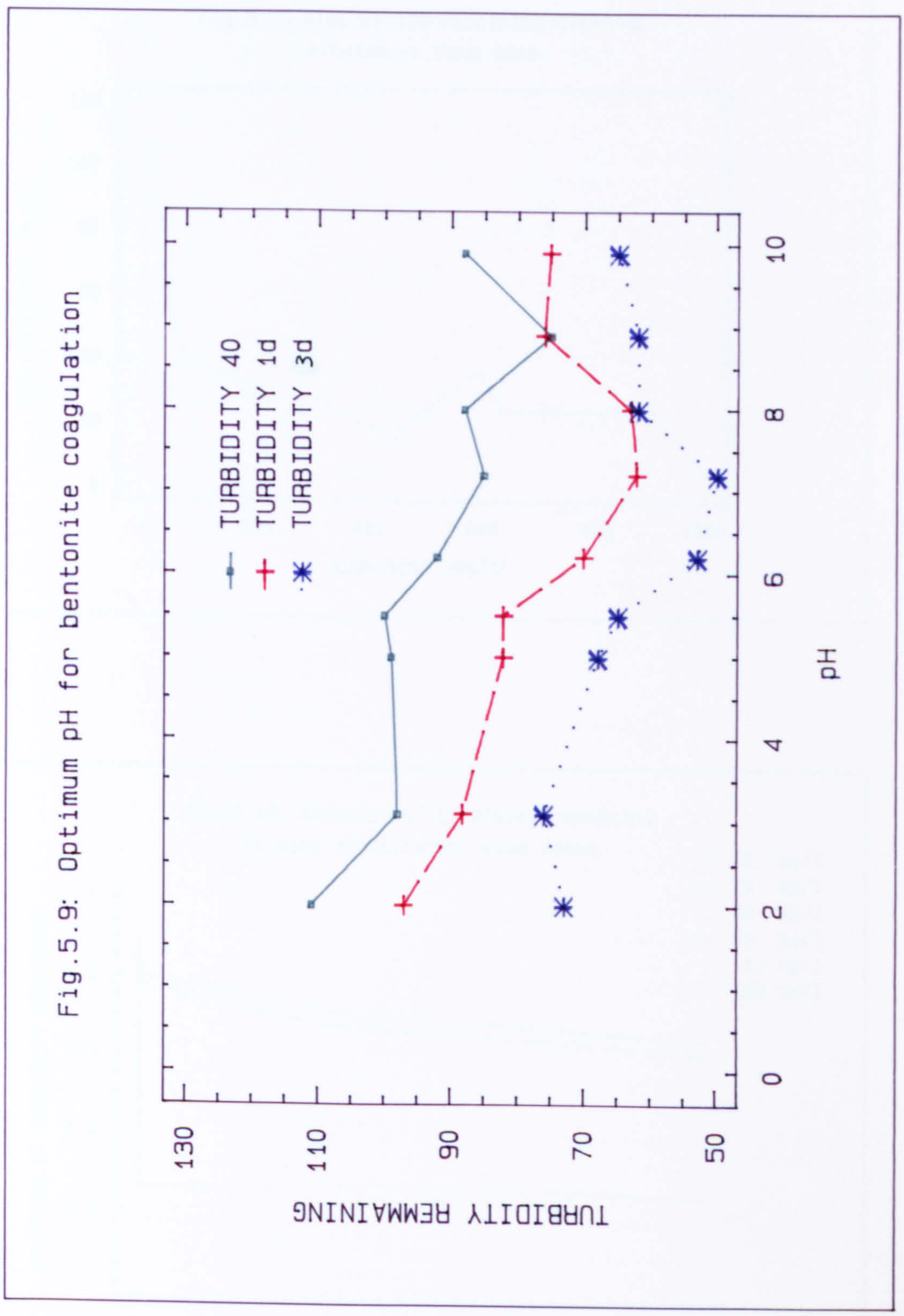


Fig.5.9: Optimum pH for bentonite coagulation
TURBIDITY 40
TURBIDITY 1d
TURBIDITY 3d
TURBIDITY REMAINING
pH
0
2
4
6
8
10
50
70
90
110
130

Fig.5.10 Plot of TSS remaining after 40 minutes vs alum dose

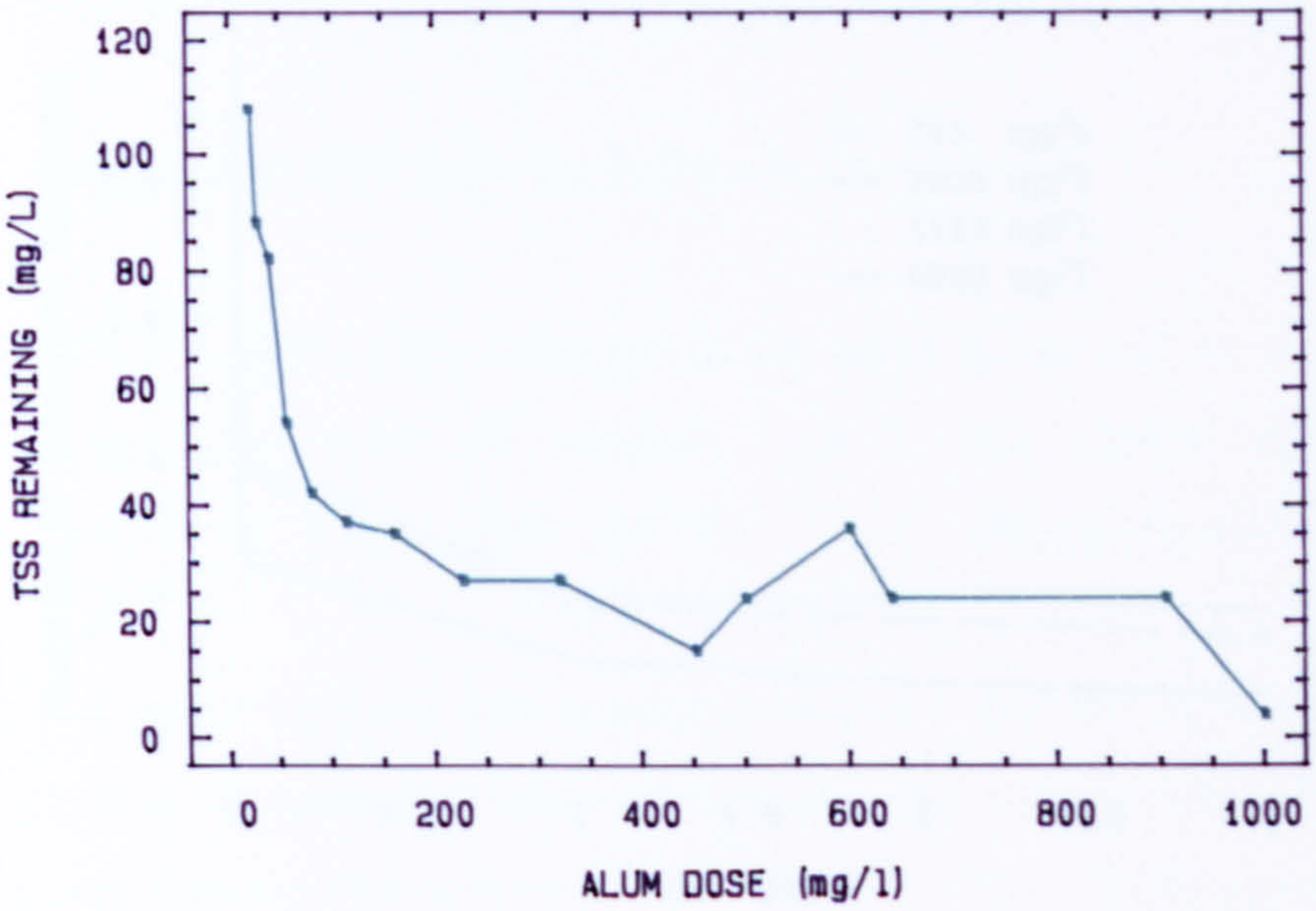


Fig.5.11: Fraction of turbidity remaining vs time at different alum doses

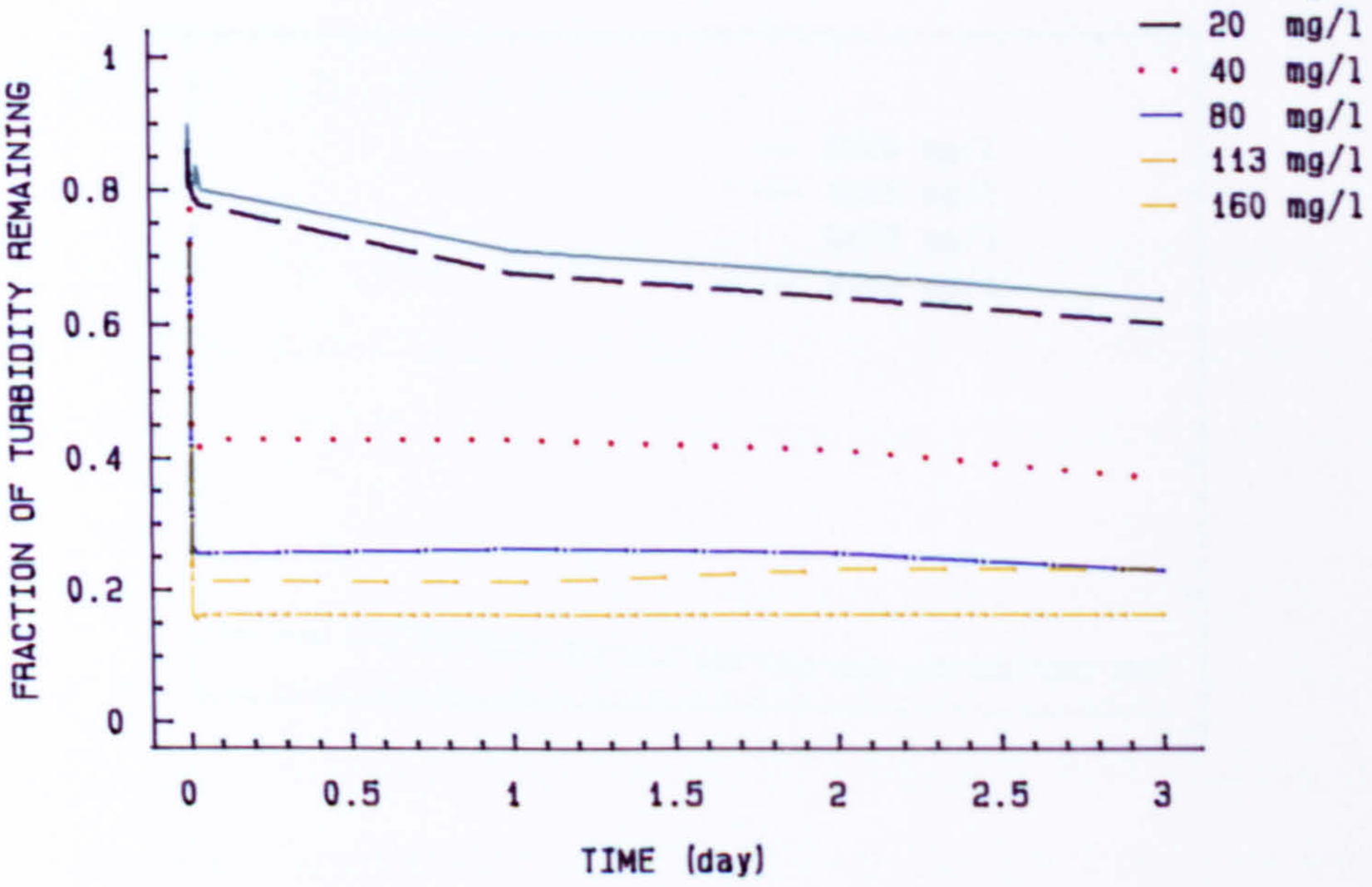


Fig.5.12a: Fraction of turbidity remaining
vs time at various kaolinite doses

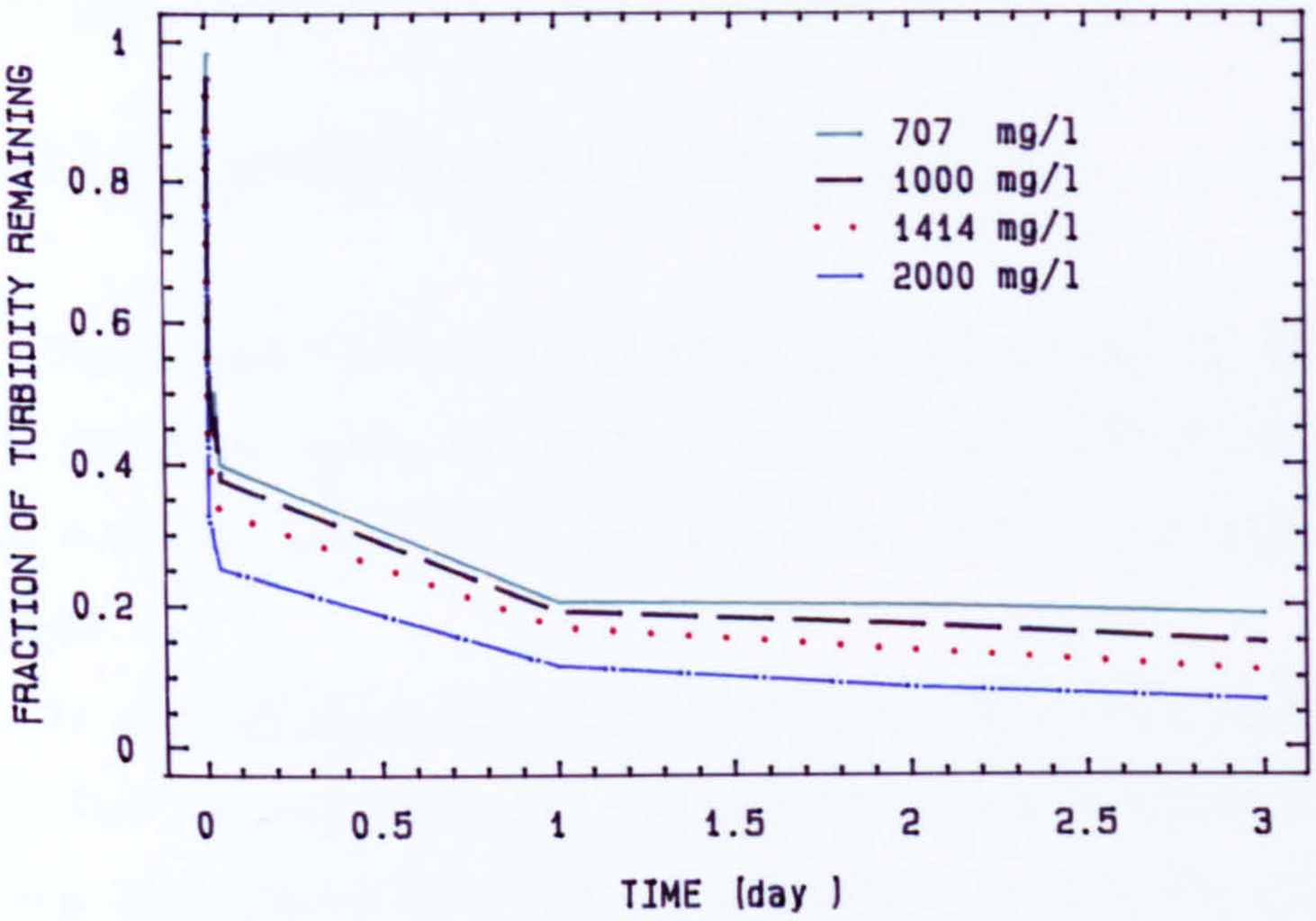
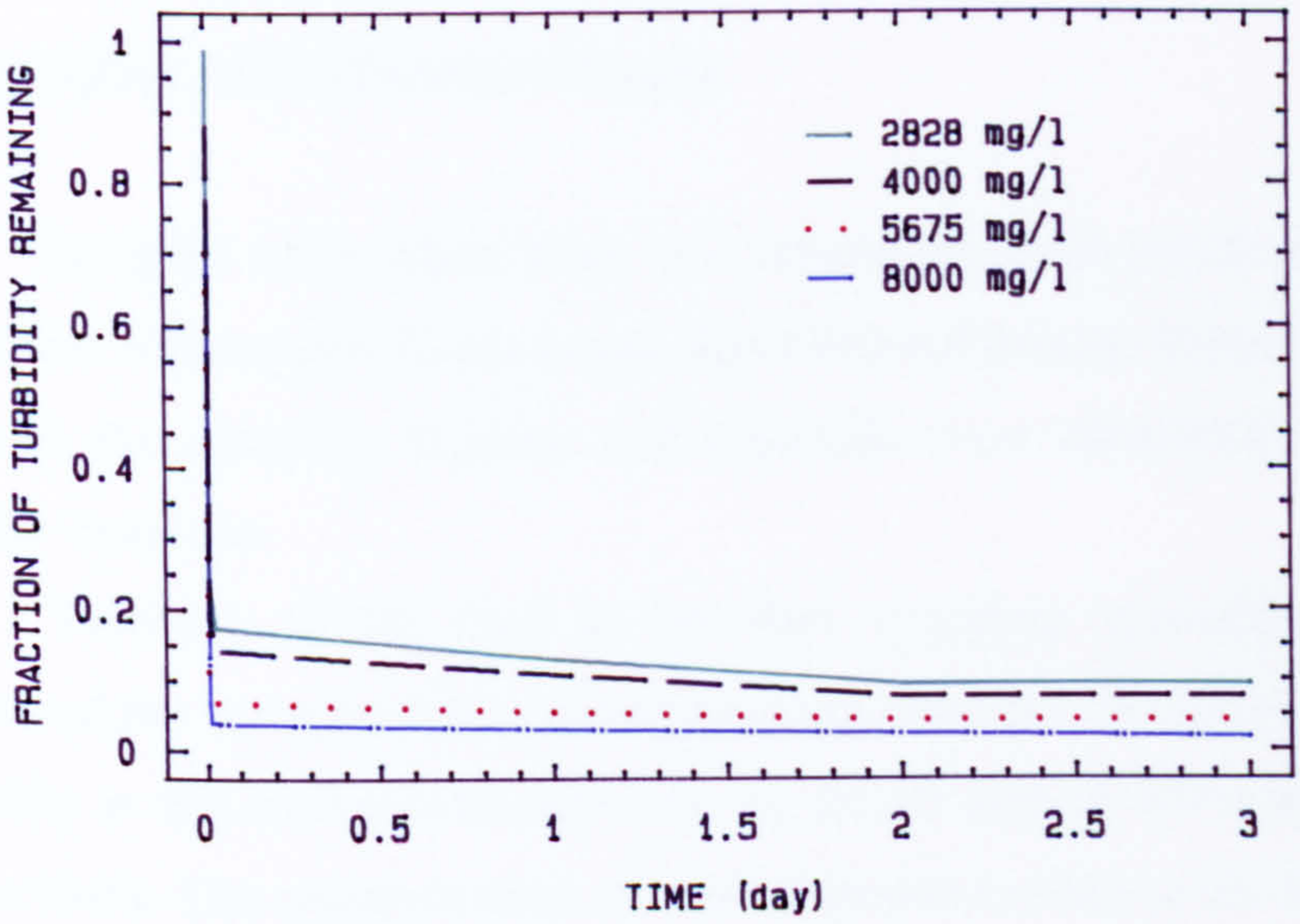


Fig.5.12b: Fraction of turbidity remaining
vs time at different kaolinite doses



A decrease of fraction of turbidity remaining with increase of coagulant dose and settling time was also noticed with bentonite applied as a primary coagulant (Fig. 5.13a, 5.13b).

Contrary to the alum results, both kaolinite and bentonite did not show inferior performance at very high coagulant doses.

5.3.3 Removal of Nutrients by Different Coagulants

BOD_5 and COD concentrations were measured in the samples taken during jar tests after 40 minutes and 3 days. These measurements are summarized in Table (5.6) for the three coagulants alum, kaolinite and bentonite.

In general, alum and bentonite showed that BOD_5 and COD decreased as the dose of coagulant increased within the tested limits. For kaolinite, an optimum dose may be observed. The concentration of BOD_5 and COD increased when the kaolinite dose was less or more than this optimum. For the three coagulants, Table (5.7) presents a summary of the results for soluble and total phosphorus removal. Also, ammonia and total Kjeldahl nitrogen concentrations are shown in this Table. After 3 days of settling, removal of both phosphorus and nitrogen improved when compared with 40 minutes settling results.

5.4 Crossflow Microfiltration Results

Samples were taken from the laboratory-scale ponds and investigated under the microscope. It was found that *Chlamydomonas* formed the majority of algae in the samples. *Euglena and Chlorella* were identified but in relatively smaller numbers.

Turbidity of the feed in the four crossflow microfiltration runs was measured for samples taken throughout the duration of each run. The average turbidity in the feed was found to be 15, 20, 25 and 16 NTU for the four runs, respectively. The concentration of total suspended solids in the feed for the third run was 125 mg/l.

Fig.5.13a: Fraction of turbidity remaining
vs time at different bentonite doses

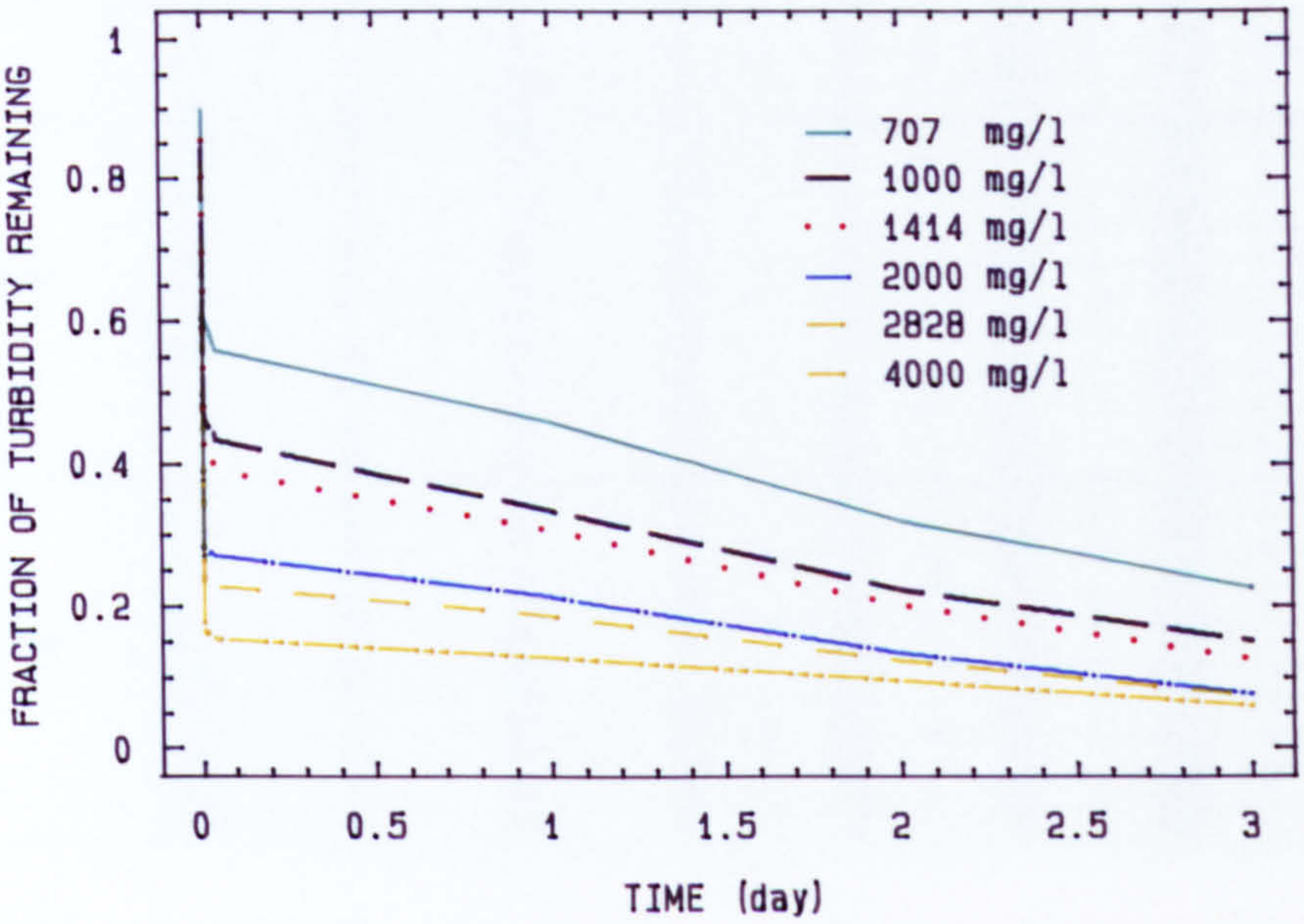


Fig.5.13b: Fraction of turbidity remaining
vs time at different bentonite doses

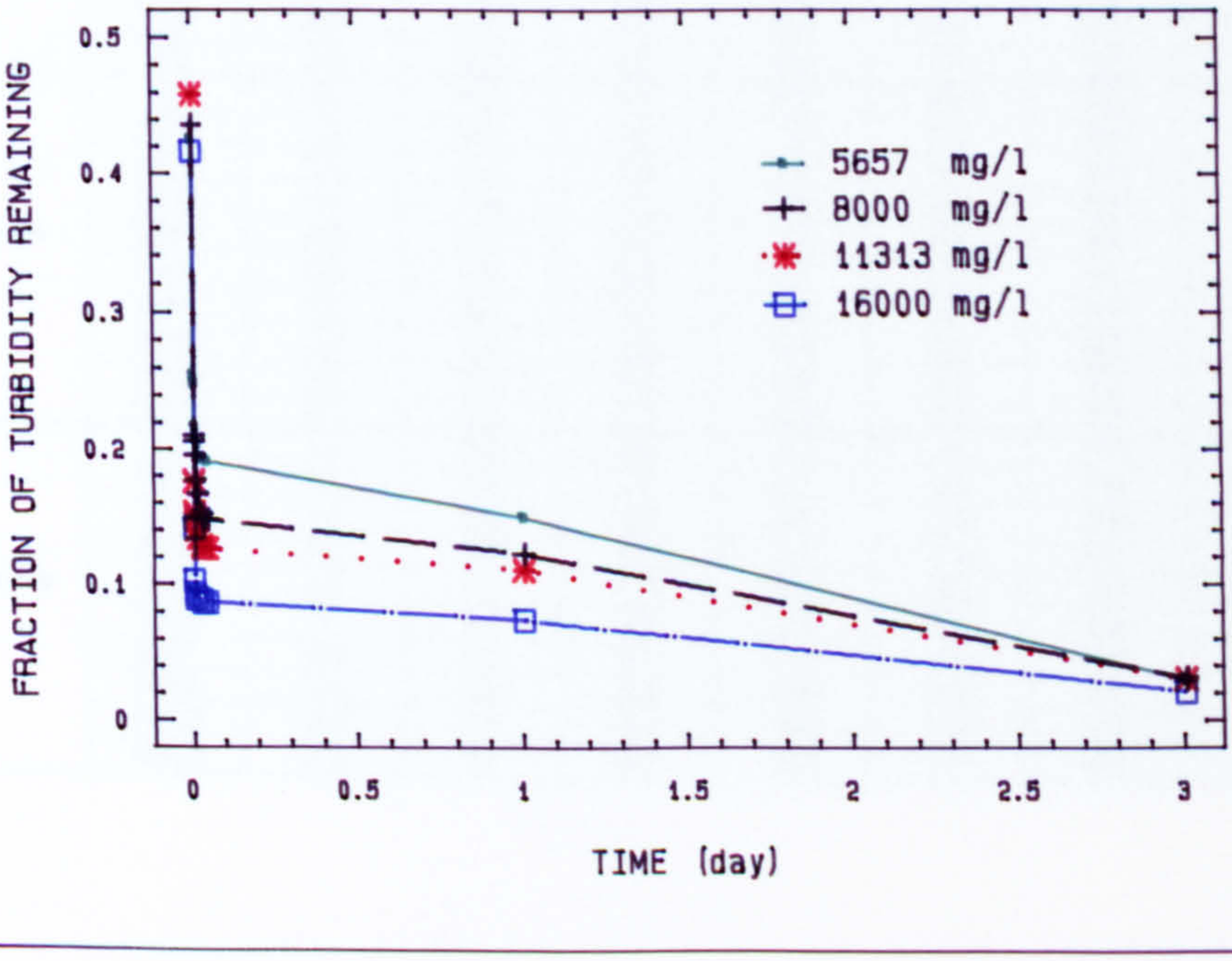


Table 5.6: BOD and COD concentration in the supernatant of the jars for different coagulants and doses

COAGULANT	DOSE mg/l	40 minutes settling		3 days settling		40 minutes settling		3 days settling	
		BODs mg/l	BODt mg/l	BODs mg/l	BODt mg/l	CODs mg/l	CODt mg/l	CODs mg/l	CODt mg/l
alum	10	51	67	20	23	196	312	170	200
	20	28	33	13	23	164	326	135	250
	40	20	30	10	23	126	226	110	190
	80	10	15	10	13	116	136	102	115
	113	5	12	5	8	109	133	85	100
	160	5	12	5	8	104	116	76	95
kaolinite	707	43	60	43	50	155	206	151	156
	1000	40	45	38	41	123	266	117	176
	1414	35	45	34	42	100	250	91	146
	2000	38	50	38	42	125	266	100	146
	2828	40	53	40	43	130	298	117	166
	4000	51	81	36	40	124	166	115	151
	5657	48	77	33	37	124	246	120	225
	8000	45	75	36	40	104	216	95	175
	11314	44	73	36	40	131	166	111	143
	16000	46	64	36	40	111	186	103	164
bentonite	707	35	80	23	28	173	250	180	210
	1000	33	78	19	20	280	290	127	150
	1414	27	72	18	27	127	250	120	180
	2000	31	70	18	20	140	210	120	170
	2828	38	62	18	20	113	220	103	136
	4000	27	43	18	20	100	200	96	116
	5657	16	40	16	20	87	310	85	126
	8000	13	37	13	20	77	260	73	106
	11314	13	37	13	18	73	240	73	106
	16000	9	33	8	18	73	230	73	106

Table 5.7: Nutrients concentration in the supernatant of jars for different coagulants and doses

COAGULANT	DOSE mg/l	40 minutes settling		3 days settling		40 minutes settling		3 days settling	
		PO4 mg/l-P	TOTAL P mg/l-P	PO4 mg/l-P	TOTAL P mg/l-P	NH3 mg/l-N	TKN mg/l-N	NH3 mg/l-N	TKN mg/l-N
alum	10	4.4	5.0	4.4	4.5	3.7	6.2	2.8	5.2
	20	4.3	5.0	4.3	4.4	3.7	6.0	2.6	5.1
	40	4.2	4.8	4.2	4.3	3.4	6.0	2.6	5.1
	80	3.5	4.0	3.3	3.8	3.4	5.9	2.3	4.7
	113	3.4	3.9	3.2	3.5	3.2	5.7	2.0	4.4
	160	2.9	3.5	2.3	2.6	2.7	5.2	1.3	3.7
kaolinite	707	4.5	5.3	4.3	5.0	4.3	6.8	4.3	6.7
	1000	4.5	5.3	4.2	4.9	4.0	6.5	4.0	6.4
	1414	4.4	5.2	3.6	4.3	3.1	5.6	2.9	5.4
	2000	4.1	4.9	4.0	4.3	3.3	5.6	3.1	5.2
	2828	3.8	4.6	3.5	4.0	3.4	5.6	3.0	5.2
	4000	3.8	4.6	3.5	4.0	3.6	5.6	3.0	5.4
	5657	3.0	3.8	2.7	3.3	4.0	6.0	3.0	5.4
	8000	2.3	3.1	2.3	2.8	4.8	6.6	3.2	5.4
	11313	2.5	3.2	2.3	2.8	4.5	6.4	3.5	6.0
	16000	2.5	3.2	2.2	2.8	4.5	6.3	4.0	6.1
bentonite	707	4.7	5.0	4.6	4.7	4.3	6.8	4.2	6.5
	1000	4.7	5.0	4.6	4.7	3.8	6.3	3.7	6.2
	1414	4.7	5.2	4.5	4.6	3.0	5.5	2.9	5.3
	2000	4.5	5.2	4.3	4.4	2.8	5.3	2.8	5.1
	2828	4.3	4.9	4.2	4.3	2.8	5.2	2.7	5.0
	4000	4	4.7	3.8	4.2	2.7	5.0	2.4	4.7
	5657	3.8	4.4	3.4	4.0	2.0	4.4	1.9	4.1
	8000	3.4	4.0	3.3	3.9	1.9	4.3	1.9	3.9
	11313	3.3	4.0	3.2	3.8	1.8	4.2	1.7	3.8
	16000	3.2	3.7	3.0	3.2	1.7	4.0	1.3	3.8

The characteristics of the permeate samples collected during each run are shown in Table (5.8).

Table 5.8: Characteristics of the permeate during crossflow microfiltration runs.

Time hr:min	Run 1		Run 2		Run 3 **		Run 4	
	Turb*	Flux+	Turb	Flux	Turb	Flux	Turb	Flux
00:00	4.5	444	14	190	13	346	8.9	137
00:05	4.2	126	13	118	13	189	8.8	127
00:10	4.2	--	13	75	13	128	8.3	107
00:15	4.2	121	12	63	13	118	8.4	107
00:20	4.0	--	12	59	13	94	8.6	--
00:30	4.0	120	12	47	13	81	8.6	90
00:45	3.8	118	12	41	13	79	8.6	84
01:00	4.0	91	12.5	40	13	78	8.6	83
01:30	4.0	78	14	40	13	75	7.5	65
02:00	3.8	75	14.5	39	13	75	7.8	64
03:00	4.1	70	13.5	36	13	74	7.5	42
04:00	3.9	70	13	35	13	73	7.2	39

** an alum dose of 10 mg/l was added to the feed in this run

* Turb=turbidity, NTU

+ Flux, $l/m^2.hr$

CHAPTER 6

DISCUSSION

6.1 Mechanisms Of Removal Of Algae in Upflow Rock Filters.

Inconsistencies in the results obtained for different rock filters has revealed a lack of agreement over the mechanisms and design criteria for this process (Middlebrooks, 1988). In sand filtration, different mechanisms have been recognized and investigated. However, in rock filtration for algae separation it has been assumed that sedimentation is the main active mechanism of removal (Williamson and Swanson, 1979). In the present work, further insight into the mechanisms involved in algae capture by rock filters has been the objective.

6.1.1 Transport Mechanisms

In Chapter 2, different transport mechanisms were represented by dimensionless groups (Eq. 2.1 to Eq. 2.5). These dimensionless groups have been calculated and are presented in Table (6.1) for algae as the particle to be removed and rock as the filter medium.

Calculations were made using the following data (Eppley et. al., 1967; Metcalf and Eddy, 1979; Palmer, 1980; APHA et al., 1985; and Boney, 1989):

Temperature = 9-34 °C

ρ_w = 999.7-994.3 kg/m^3

μ = 1.307×10^{-3} - 0.740×10^{-3} N.s/m^2

$\bar{\kappa}$ = Boltzman's constant = 1.381×10^{-23} J/k

algal species : *chlamydomonas, euglena, phacus, oscillatoria, chlorella, phytoconis, micractinium, anacystis.*

d_p = 2 - 25 μm

ρ_s = 1.01 - 1.1 g/cm^3

Table 6.1: Transport mechanisms evaluation

GROUP	MECHANISMS	APPROACH VELOCITY v (m/d)	VALUE OF THE GROUP		
			d_c=1 cm, n=0.37	d_c=5 cm, n=0.42	d_c=10 cm, n=0.47
$S= g(\rho_s-\rho_w)/18\mu v_i$	settling	0.2 1.0 2.0 3.0 4.0	$2.7x10^{-3}$-7.8 $5.5x10^{-4}$-1.6 $2.7x10^{-4}$-0.8 $1.8x10^{-4}$-0.5 $1.4x10^{-4}$-0.4	$3.1x10^{-3}$-8.8 $6.2x10^{-4}$-1.8 $3.1x10^{-4}$-0.9 $2.1x10^{-4}$-0.6 $1.6x10^{-4}$-0.44	$3.4x10^{-3}$-9.7 $6.8x10^{-4}$-1.9 $3.4x10^{-4}$-1.0 $2.3x10^{-4}$-0.64 $1.7x10^{-4}$-0.48
$I= d_p/d_c$	interception	ALL	$2x10^{-4}$-$2.5x10^{-3}$	$4x10^{-5}$-$5x10^{-4}$	$2x10^{-5}$-$2.5x10^{-4}$
$P^{-1}= \kappa T/3\pi\mu d_p v d_c$	diffusion	0.2 1.0 2.0 3.0 4.0	$5.5x10^{-7}$-$1.3x10^{-5}$ $1.1x10^{-7}$-$2.6x10^{-6}$ $5.5x10^{-8}$-$1.3x10^{-6}$ $3.6x10^{-8}$-$8.8x10^{-7}$ $2.7x10^{-8}$-$6.6x10^{-7}$	$1.1x10^{-7}$-$2.6x10^{-6}$ $2.2x10^{-8}$-$5.2x10^{-7}$ $1.1x10^{-8}$-$2.6x10^{-7}$ $7.3x10^{-9}$-$i.8x10^{-7}$ $5.5x10^{-9}$-$1.3x10^{-7}$	$5.5x10^{-8}$-$1.3x10^{-6}$ $1.1x10^{-8}$-$2.6x10^{-7}$ $5.5x10^{-9}$-$1.3x10^{-7}$ $3.6x10^{-9}$-$8.7x10^{-8}$ $2.7x10^{-9}$-$6.6x10^{-8}$
$E= \rho_s d_p^2 v/18\mu d_c$	inertia	0.2 1.0 2.0 3.0 4.0	$-4.x10^{-11}$-$1.2x10^{-8}$ $2.x10^{-10}$-$6.0x10^{-8}$ $4.x10^{-10}$-$1.2x10^{-7}$ $6.x10^{-10}$-$1.8x10^{-7}$ $8.x10^{-10}$-$2.4x10^{-7}$	$8.x10^{-12}$-$2.4x10^{-9}$ $4.x10^{-11}$-$1.2x10^{-8}$ $8.x10^{-11}$-$2.4x10^{-8}$ $1.2x10^{-10}$-$3.6x10^{-8}$ $1.6x10^{-10}$-$4.8x10^{-8}$	$4.x10^{-12}$-$1.2x1^{-9}$ $2.x10^{-11}$-$6.0x10^{-9}$ $4.x10^{-11}$-$1.2x10^{-8}$ $6.x10^{-11}$-$1.8x10^{-8}$ $8.x10^{-11}$-$2.4x10^{-8}$
$R_e^{-1}= \mu/vd_c\rho_w$	hydrodynamics	0.2 1.0 2.0 3.0 4.0	32.0-56.8 6.4-11.4 3.2-5.7 2.1-3.8 1.6-2.8	6.4-11.4 1.3-2.3 0.6-1.1 0.4-0.8 0.3-0.6	3.2-5.7 0.6-1.1 0.3-0.6 0.2-0.4 0.16-0.3

(I) Sedimentation

In Eq. 2.1, Ison and Ives (1969) and Ives (1975a) used the approach velocity, v, to find the effect of gravity. They suggested the ratio v_s/v where v_s is Stoke's settling velocity of the particle. Ives (1975a) argued that all liquid velocities inside the pore, such as velocity in the direction of flow, velocity perpendicular to the flow, velocity along a stream line and velocity parallel to the surface of the collector, are proportional to interstitial velocity v_i and, therefore, proportional to v. However, using the ratio v_s/v does not show the effect of the size of the media. Hence, in this work the ratio v_s/v_i was used in the calculation in Table (6.1) so as to show the variation in the effect of gravity with respect to change in rock size. Interstitial velocity is related to approach velocity by the porosity n, where $v_i = v/n$.

In Table (6.1), when the settling dimensionless group S was calculated, smaller d_p, ρ_s and larger μ and v_i were inserted into the group to calculate the lower limit (e.g 2.0×10^{-3} for $d_c = 1$ cm). Exactly the reverse was used to calculate the higher limit (e.g 7.8 for $d_c = 1$cm). This leads to the conclusion that settling is not always the main algae removal mechanism, as was suggested by Williamson and Swanson (1979). In fact, settling becomes very active as a mechanism for larger cells (25 μm), especially at low filtration rates. Even at higher filtration rates (4 m/d), settling is still active for large cells.

Variation of temperature changes the value of the dimensionless group S that represents settling (Eq. 2.1). For example, at a filtration rate of 1 m/d and $d_p = 25$ μm, S increases from 6.104 to 11.4 when temperature is increased from 9 to 34°C. Although this may indicate that settling is enhanced, it should not be interpreted that removal efficiency is doubled due to an increase in temperature from 9 to 34°C . Ives (1975a) stated that this group S should be understood as a relative dimensionless group that indicates the importance of settling as a mechanism of removal but not as a basis for judging the efficiency of the filtration process.

An important note on the values of S drawn from Table (6.1) is that they increase with increase of rock size. Smaller rock size gives higher interstitial

velocity, hence settling will be enhanced in the filter with larger rock size. Stated differently, larger rock media contains larger pores which offer more chance for sedimentation to occur.

(II) Interception

The value of the dimensionless group I (Eq. 2.2) in Table (6.1) is lower than usually encountered in water filtration [2×10^{-4}-1×10^{-1} , Ives(1975a)]. This does not suggest that interception is not taking place at all during algae removal by rock filtration. However, its relative importance may be small because of the fact that the media employed has a relatively large size and the solids being removed are relatively small. Again it is noticed that interception is more active in the filters with smaller rock size (Table 6.1). In fact, interception was shown to be of little importance in particle removal during sand filtration where filter grains are much smaller than those encountered in rock filters(Ison and Ives,1969). Thus, it is unlikely that interception would have an important role in the process of rock filtration.

(III) Diffusion

It has been shown that algal suspensions have colloidal properties. In discrete form, algal cells possess a negative charge, undergo chemical reactions on the surface and make stable suspensions. This may indicate that diffusion is an active mechanism in the removal of algae in rock filter pores. However, it has been found that diffusion is only effective as a mechanism of removal for particles less than 1 μm in diameter (O'Melia,1985).

Algal cells, except filamentous forms, range in size between 5 and 50 μm (Tenney et al., 1969). In this work the range was 2-25 μm, with the majority between 10 and 25 μm. This size is greater than the colloidal range (<1 μm) which would suggest that diffusion may not be considered an important mechanism of removal of algae in rock filters.

(IV) Inertia

Values of E (Eq. 2.4), that represent inertia, range between 1.7×10^{-9} - 1.5×10^{-3} for clay particles removed by rapid sand filters. This mechanism was considered as negligible in terms of collection efficiency (Ison and Ives, 1969; Ives, 1970 and 1975a). Values of E calculated for rock filters (Table 6.1) in this research are on the lower side of the range reported above for rapid sand filtration. Hence, inertia makes almost insignificant contribution to the removal of algae in rock filtration, although it is relatively more important in the filter with smaller pores (d_c = 1 cm) compared with the other two rock filters (d_c=5cm, d_c=10cm).

(V) Hydrodynamic Action

Eq. 2.5 does not show the dependence of hydrodynamic action on particle size. Ives (1975a) suggested that all variables, such as the velocity of the particle relative to the liquid v_p , the angular velocity of a rotating particle ω, and the change in fluid flow due to pore size sequences, are all represented by Eq. 2.5. To express the dependence on particle size, d_p, he introduced an equation which he described as entirely speculative.

However, with constant d_p/d_c, Ison and Ives (1969) showed that increasing Reynolds number decreased removal efficiency. Table (6.1) shows clearly that filters with large filter media have lower R_e^{-1} values. Thus, the hydrodynamic effect is less effective in larger pores. This may be explained by the fact that the hydrodynamic mechanism is attributed mainly to viscous forces, μ. Thus, decreasing the inertia forces ($vd_c\rho_w$) would make viscous forces a more dominant and produce a more random drift. The same applies if R_e was written with d_p instead of d_c. Smaller particles will be subjected to less inertia forces and their opportunity to move across the streamlines is more probable.

6.1.2 Combined Action

Fig. (5.4) shows clearly that algal removal efficiency improved with decrease in hydraulic loading or approach velocity,v. So, it may be concluded that mechanisms that are enhanced at lower approach velocities are more dominant in the process of algae removal by rock filters. Thus, inertia action may be less important in this process. Settling, diffusion, and hydrodynamics are expected to be more active during the process of algae removal by rock filters.

However, to learn more about the relative importance of each of the mechanisms listed in Table (6.1), Ives' Eq. 2.10 may be solved for the values of α, β, γ and δ.

Ives (1975a) suggested that a variation of temperature between 5 and 25°C will make a negligible change in thermodynamic energy ($\bar{\kappa}T$) and water density ρ_w. Increasing temperature from 5-25°C produces an increase of 7% in ($\bar{\kappa}T$). An increase from 9-34°C, which is the range encountered in this research, results in a 8.86% increase in ($\bar{\kappa}T$), which is still a small variation. The density of the liquid, ρ_w, only decreases by 0.5% when temperature is increased from 9-34°C. This leaves four variables in the experiments: d_p, μ, d_c and v .

Table (6.2) shows the dominant species of algae and their corresponding average diameters, d_p. Also, prevailing temperatures and related dynamic viscosities, μ, are shown. At each combination of d_p, ρ, d_c and v, four simultaneous equations are required for the evaluation of α, β, γ and δ. Experimental set No.1 has a particle diameter, d_p, of 12 μm and an average temperature of 12.4°C ($\mu = 1.227\times10^{-3}$ N.S/m^2). An average temperature of 16°C ($\mu = 1.112$ N.S/m^2) and $d_p = 15$ μm represent experimental set 2. For each d_c size in experimental set 1, d_p, μ, d_c, ρ_s, $\bar{\kappa}$, T, and ρ are constants. Therefore, each one of the dimensionless groups $1/p_e$, S and $1/R_e$ is a linear combination of I, $1/p_e$, S and $1/R_e$. Hence, only two independent equations can be produced from experimental set 1 for Λ, given by Eq. 2.10 or 2.11. The same argument applies for experimental set 2.

In experimental set 3, it is difficult to choose a characteristic particle diameter and temperature because of the diversity of algal species and the variation of temperature through the experimental set. Since only four equations

Table 6.2: Some influent characteristics

Experi-mental set No.	Hydraulic loading (m3/m3.d)	Approach velocity v (m/d)	Monthly average temperature (°C)	Dynamic viscosity x10 E-3 ($N.s/m^2$)	Dominant * algal species	Diameter of particle (μm)
1	0.1	0.2	13.5	1.193	*chlamydomonas*	12
1	0.5	1.0	13	1.210	*chlamydomonas*	12
1	1.0	2.0	13	1.210	*chlamydomonas*	12
1	1.5	3.0	11.5	1.257	*chlamydomonas*	12
1	2.0	4.0	11.5	1.257	*chlamydomonas*	12
2	0.1	0.2	15.5	1.122	*chlamydomonas, euglena*	10-25
2	0.5	1.0	15.5	1.122	*chlamydomonas, euglena*	10-25
2	1.0	2.0	15.5	1.122	*chlamydomonas, euglena*	10-25
2	1.5	3.0	20	1.007	*chlamydomonas, euglena*	10-25
2	2.0	4.0	15.5	1.122	*chlamydomonas, euglena*	10-25
3	0.1	0.2	30.5	0.789	*euglena, micractinium, anacystis*	4-25
3	0.5	1.0	22.5	0.946	*chlamydomonas, euglena, phacus, oscillatoria*	4-50
3	1.0	2.0	22.5	0.946	*chlamydomonas, euglena, phacus, oscillatoria*	4-50
3	1.5	3.0	30.5	0.789	*euglena, micractinium, anacystis*	7-25
3	2.0	4.0	25	0.886	*euglena, chlorella, phytoconis*	6-25

* Source: RSS (1991)

are needed to find the values of α, β, γ, δ, the equations formed from groups one and two are enough for this purpose. Ives (1975a) did not count the constant in Eq. 2.10 as one of the unknowns to be determined. This may be due to the fact that determining the constant will only affect the absolute values of α, β, γ and δ but the relative importance of the mechanisms, represented by these four constants, will not be affected.

Multiple regression was used to solve thirty equations set at each hydraulic loading for the three rock filters using the two experimental sets one and two (see Appendix-B). The results were : const.=3.7×10^{-7} with significance level (SL) of 0.00, α=-0.604 (SL=0.00), β=-0.336 (SL=0.10), γ=0.473 (SL=0.00) and δ=0.181 (SL=0.10).

The negative values of both α and β are statistically significant but do not agree with the physical concepts of interception and diffusion. The form of Eq. 2.10, using Λ to describe the removal efficiency of filters, is thought to be responsible for such technically invalid results.

The value of Λ represents the fraction of suspended solids held in a filter layer one grain diameter thick. Comparison based on Λ may be misleading if the filters under consideration are of different rock sizes; although a filter of smaller rock size may be more effective, the material retained in a layer one rock diameter thick may be less than the amount of suspended solids captured in another larger layer in a second filter which is less efficient but its rock size is bigger. Stated differently, λ_1 may be less than λ_2 for two different filters while Λ_1 may be greater than Λ_2 for these same two filters. In this case, solving Eq. 2.10 using Λ will result in negative values of any one of α, β, γ and/or δ. Negative values of these constants contradict the physical concept of interception, diffusion, gravitational settling and hydrodynamic forces. Furthermore, dividing Eq. 2.10 by d_c will result in different constants, on the right hand side of the equation, for filters of different grain sizes. This means that Eq. 2.10 is not a general equation that can be used to judge the removal efficiency of filters of different media sizes. The use of Λ by Ison and Ives (1969) may explain why they obtained negative values of α, namely $(d_p/d_c)^{-2.3}$. To explain such a result they gave three reasons which they described as speculative.

Rewriting Eq. 2.6 as $\lambda=(-\Delta C/\Delta x)/C$ describes λ as the fraction of suspended solids removed per unit filter height. In Eq.2.10, Λ is dimensionless and is intuitively more appealing than λ, which has dimensions of L^{-1}. However, the results gave a technically invalid solution when Λ was used in Eq. 2.10. Thus, it may be hypothesised that λ itself is a function of interception, diffusion, settling and hydrodynamics as shown in Eq. 6.1:

$$\lambda = A(interception)\,(diffusion)\,(ettling)\,(hydrodynamics) \quad ..(6.1)$$

Then Eq. 6.1, 2.1, 2.2, 2.3 and 2.4 yield Eq. 6.2:

$$\lambda = A\left(\frac{d_p}{d_c}\right)^{\alpha}\left(\frac{\overline{kT}}{3\mu d_p v d_c}\right)^{\beta}\left(\frac{g(\rho_s-\rho_w)}{18\mu v}\right)^{\gamma}\left(\frac{\mu}{v d_c \rho_w}\right)^{\delta} \quad ...(6.2)$$

In Eq. 6.1 and 6.2, A is a proportionality constant with the same dimensions as λ.Solving Eq.6.2 in the same way that Eq.2.10 was solved (Appendix- B),resulted in:Log A=-0.581(SL=0.66),A=0.262, α=0.128 (SL=0.30),

β=-0.086(SL=0.71), γ=0.226(SL=0.07) and δ=0.177(SL=0.10). Although the value of β is negative, there is a very low degree of confidence that its value is significantly different from zero. Also, the confidence is as low as 34% that Log A is different from zero, which means that A can be taken equal to 1.

The positive value of α in this work shows that removal of algae by interception increases with increase of d_p/d_c ratio. This agrees with the results of Yao et al. (1971) and O'Melia and Ali (1978). They found that the efficiency of removal of suspended solids is proportional to $(d_p/d_c)^2$. However, the value of α in this work is very small and is not significantly different from zero, which indicates that interception is not an important mechanism of removal of algae in rock filters.

The fact that $(\alpha-\beta+2\gamma)$ is positive indicates two things. First, removal of algae as a whole is improved with increase of particle diameter. Second, it indicates that mechanisms that act most effectively with higher d_p values, are the dominant mechanisms in algae capture by rock filters.

Hence, settling is important. In fact, it possesses the highest power effect, i.e the value of γ =0.226 is the largest with 93% confidence that it is different from zero, which indicates that settling is the most significant mechanism of removal of algae in the pores of rock filters receiving effluent from facultative ponds. This supports experimentally the assumption made by Swanson and Williamson (1980), also Williamson and Swanson (1979). They suggested that sedimentation is the basic mechanism of removal of algae in rock filters.

An important finding is that efficiency of algae removal is proportional to $\mu^{\beta+\gamma-\delta}$ or to $\mu^{-0.05}$, which indicates that the variation in temperature is not a much significant factor in algae removal by rock filtration when its physical effect on viscosity is the matter of concern. This shows the importance of not judging the effect of certain variables on removal when one mechanism is assumed as the only one operating, without supporting experimental evidence. In the previous Section, where the settling effect was analyzed separately, temperature seemed to have a pronounced effect on removal by changing viscosity. However, the effect of viscosity may be lessened if other mechanisms are acting, as has been proved in this work.

The analysis carried out in the course of this research shows clearly that other mechanisms are also important. The value of δ=0.177 reveals that hydrodynamic action is also active in algae removal.

The relatively small values of α and β indicate that neither interception nor diffusion is effective as a mechanism of algae removal. This is in harmony with the findings of Yao et al. (1971) that diffusion is only effective for particles less than 1μm in diameter.

6.1.3 Other Mechanisms

(I)Straining

Because the diameter of the media involved in rock filter is very large compared with the size of algae being removed, straining does not have any importance as a mechanism of removal of algae by rock filtration.

(II)Flocculation

Two types of flocculation are relevant to algae removal in rock filters. The first is autoflocculation caused mainly by the production of exocellular polymers. The second type of flocculation is due to the differential velocity in the pores.

Stutz-McDonald and Williamson (1975) stated that autoflocculation of algae in ponds is only observed at pH>11 and it may not be important as a mechanism of algae removal in rock filters. On the other hand, Golueke and Oswald (1965) found that pH=3 represents the optimum for algae settling without coagulants being added.

In this work, pH ranged between 7 and 8.4, which may indicate that autoflocculation did not take place during rock filtration. However, autoflocculation at pH>11 is irreversible and is due to lysis of cells (Foess and Borchardt, 1969). Autoflocculation may occur at other pH values due to secretion of exocellular anionic polymers. The work of Pavoni et al. (1972) on the coagulation of kaolinite suspensions (isoelectric point = 2) with extracted bacterial polymer showed that coagulation occurred in the pH range 6-11. Higher pH values were associated with higher polymer concentrations. The maximum production of these polymers is during endogenous respiration and/ or the limited growth period (Pavoni et al., 1972; Tenney et al., 1972; O'Melia, 1978a;

James and Evison, 1979). Also, it was noted by Gaudy and Gaudy (1988) that autoflocculation of a microbial population is associated with limited sources of food and energy. Haarhoff and Cleasby (1989) reported that 5% of the total organic carbon produced by algae is in the form of anionic polymers during the growth stage. During endogenous respiration, the figure goes up to 95%.

Under dark conditions in the pores of the rock filter algae would undergo endogenous respiration (Jackson, 1967). This would cause more production of the exocellular compounds which would help in autoflocculation of algae inside the filter.

Flocculation caused by the velocity gradient in the pores was employed successfully in gravel-bed flocculators for treatment of surface water after addition of coagulants (Shea et al., 1971; Culp, 1977; Schulz and Okun, 1984). Also, Ayoub and Nazzal (1988) found that flocculation in a baffled gravel packed channel flocculator improved turbidity removal in the clarifier 20% more than flocculation in a plain baffled channel. This sort of flocculation was related to the ratio of pore surface to pore volume, $(6/d_c)(1-\epsilon)/\epsilon$, in the filter pores (Ives, 1975a). On the other hand, such a mechanism was considered as of minor importance in water filtration (ASCE et al., 1971).

However, velocity gradient may be active in rock filters due to the existence of coagulants produced by algae. The rock filter will, in this case, act as a gravel bed flocculator mentioned above. This would enhance the autoflocculation process. Although orthokinetic flocculation may be unimportant for discrete particles in filter pores, it was proved experimentally that inter-particle collisions increase with increase in shear rate in the pores for suspensions destabilized by polymers (Graham, 1986, 1988).

6.2 Factors Involved In Algae Removal In Upflow Rock Filters

It was stated earlier that the mechanisms and design parameters are still uncertain in the field of algae removal by rock filters (Middlebrooks, 1988). One of the objectives of this work was to investigate the effect of certain operating variables on the performance of rock filters. These variables are discussed in the following sections.

6.2.1 Hydraulic Loading

Table (6.3) shows the effect of hydraulic loading on the total suspended solids removal efficiency of three rock filters. This effect is shown more clearly in Fig. (6.1), which indicates that, irrespective of the rock size, removal of TSS is increased with decrease in hydraulic loading in the three experimental sets, which represent different effluent characteristics in terms of algal species, algae concentration and environmental conditions.

O'Brien and McKinney (1979) did not notice any effect on effluent suspended solids concentration by raising the hydraulic loading to 2.978 $m^3/m^3.d$ for a horizontal flow rock filter at Eudora, Kansas (rock 0.95-5.08 cm), and to 2.188 $m^3/m^3.d$ for the filter with rock size 0.95-5.08 cm. However, the findings of this work are in agreement with the recommendations set by O'Brien (1975,1976) for the design of rock filters. He recommended lower flow rates in winter to compensate for the loss of efficiency in cold weather, according to his findings.

This work shows that in the filter filled with 1.0 cm average diameter rock, 78% TSS removal were achieved at a hydraulic loading of 0.1 $m^3/m^3.d$. The TSS removal efficiency of the same filter dropped to about 49% when the applied loading was raised to 2.0 $m^3/m^3.d$. The same trend is noticed for the other two filters (Fig. 6.1).

In the previous Section, where mechanisms of removal were discussed, it was shown that the filtration factor, λ, is proportional to $v^{-\beta-\gamma-\delta}$ or to $v^{-0.32}$. This also means that the overall effect of decrease in hydraulic loading is to improve removal efficiency. Also, it indicates that approach velocity is the second most important variable among other factors. It was found that λ is proportional to $\mu^{-0.05}$, $d_c^{-0.30}$, $v^{-0.40}$ and $d_p^{0.58}$.

6.2.2 Concentration of Total Suspended Solids in the Influent

This factor has been well recognized in filtration theory, as early as 1937 by Iwasaki. Later works still insert this factor in the models describing the process of filtration (see, for example, Hall, 1957; Mints, 1966; Ives, 1969;

Table 6.3: Average TSS percentage removal at different hydraulic loadings

SET No.	HYDRAULIC LOADING (m3/m3.d)	INFLOW TSS (mg/l)	EFFLUENT TTSS CONCENTRATION			TSS PERCENTAGE REMMOVAL		
			ROCK 10cm (mg/l)	ROCK 5cm (mg/l)	ROCK 1cm (mg/l)	ROCK 10cm (%)	ROCK 5cm (%)	ROCK 1cm (%)
1	0.1	248	73	62	39	70	75	84
2	0.1	159	72	59	33	55	63	79
3	0.1	184	75	63	37	59	66	80
1	0.5	154	77	69	54	50	55	65
2	0.5	183	88	77	59	52	58	68
3	0.5	192	88	77	58	54	60	70
1	1.0	138	86	79	62	38	43	55
2	1.0	152	88	80	59	42	47	61
2	repetition 1.0	192	113	100	70	41	48	64
3	1.0	217	108	96	70	50	56	68
1	1.5	227	145	136	100	36	40	56
2	1.5	188	132	119	85	30	37	55
3	1.5	255	153	148	97	40	42	62
1	2.0	211	158	144	116	25	32	45
2	2.0	159	118	105	77	26	34	51
3	2.0	178	132	119	89	26	33	50
3	repetition 1.0	293	144	129	88	51	56	70

Table 6.4: Average chlorophyll percentage removal at different hydraulic loadings

SET No.	HYDRAULIC LOADING (m3/m3.d)	INFLOW CHLRO-PHYLL(µg/l	EFFLUENT CHLOROP HYLL			CHLOROP HYLL REM OVAL (%)		
			ROCK 10cm (µg/l)	ROCK 5cm (µg/l)	ROCK 1cm (µg/l)	ROCK 10cm (%)	ROCK 5cm (%)	ROCK 1cm (%)
1	0.1	382	92	66	43	76	83	89
2	0.1	224	67	54	28	70	76	88
3	0.1	376	87	65	42	76	82	88
1	0.5	221	82	62	31	63	72	86
2	0.5	252	89	68	35	65	73	86
3	0.5	180	72	55	26	60	69	86
1	1.0	197	89	71	36	55	64	82
2	1.0	225	96	80	42	57	64	81
2	repetition 1.0	256	103	80	44	60	69	83
3	1.0	270	112	96	51	59	65	81
1	1.5	332	168	133	66	49	60	80
2	1.5	255	133	104	51	48	59	80
3	1.5	432	207	162	85	49	60	79
1	2.0	304	213	140	67	30	54	78
2	2.0	220	141	99	46	36	55	79
3	2.0	214	137	97	44	36	55	79
3	repetition 1.0	416	167	133	71	60	68	83

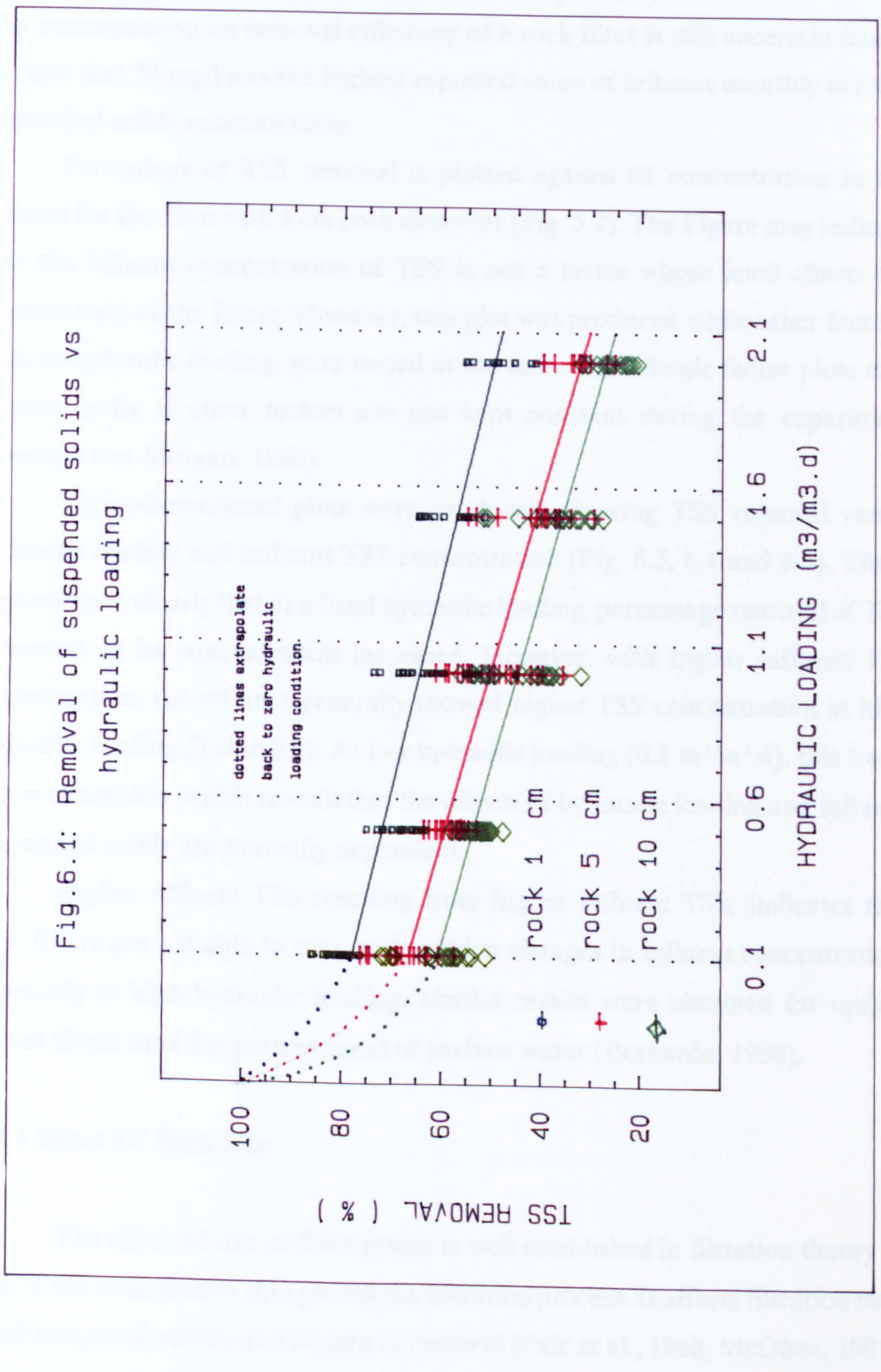


Fig.6.1: Removal of suspended solids vs hydraulic loading
dotted lines extrapolate back to zero hydraulic loading condition.
rock 1 cm
rock 5 cm
rock 10 cm
HYDRAULIC LOADING (m3/m3.d)
0.1
0.6
1.1
1.6
2.1
TSS REMOVAL (%)
100
80
60
40
20

O'Melia and Ali, 1978; O'Melia, 1985;). However, the concentration of algae in the influent to a rock filter has not been identified as a factor affecting the process of algal removal by these filters. Parker (1976) stated that the effect of TSS concentration on removal efficiency of a rock filter is still uncertain due to the fact that 78 mg/l was the highest reported value of influent monthly average suspended solids concentration.

Percentage of TSS removal is plotted against its concentration in the influent for the filter with 1 cm rock diameter (Fig. 6.2). The Figure may indicate that the influent concentration of TSS is not a factor whose level affects the performance of the filters. However, this plot was produced while other factors, such as hydraulic loading, were varied at the same time. Single factor plots may become noisy if other factors are not kept constant during the experiment (Deming and Morgan, 1988).

Three-dimensional plots were produced, showing TSS removal versus hydraulic loading and influent TSS concentration (Fig. 6.3, 6.4 and 6.5). These Figures show clearly that at a fixed hydraulic loading, percentage removal of TSS increased as its concentration increased. However, with higher influent TSS concentration, the effluent generally showed higher TSS concentration at high hydraulic loading (Table 6.3). At low hydraulic loading (0.1 $m^3/m^3.d$), this trend is less noticeable, which reveals that the effects of hydraulic loading and influent suspended solids are mutually dependent.

Higher effluent TSS resulting from higher influent TSS, indicates that rock filters are not able to cope with sudden changes in influent concentration, especially at high hydraulic loading. Similar results were obtained for upflow gravel filters used for pretreatment of surface water (Bernardo, 1988).

6.2.3 Effect Of Rock Size

The effect of size of filter grains is well established in filtration theory as one of the basic factors that govern the filtration process. It affects filtration rate, head loss, mechanisms and degree of removal (Fair et al., 1968; McGhee, 1991). However, in the field of rock filters it is not known to what degree removal of

Fig.6.2: Suspended solids removal vs.
influent suspended solids concentration for rock 1 cm

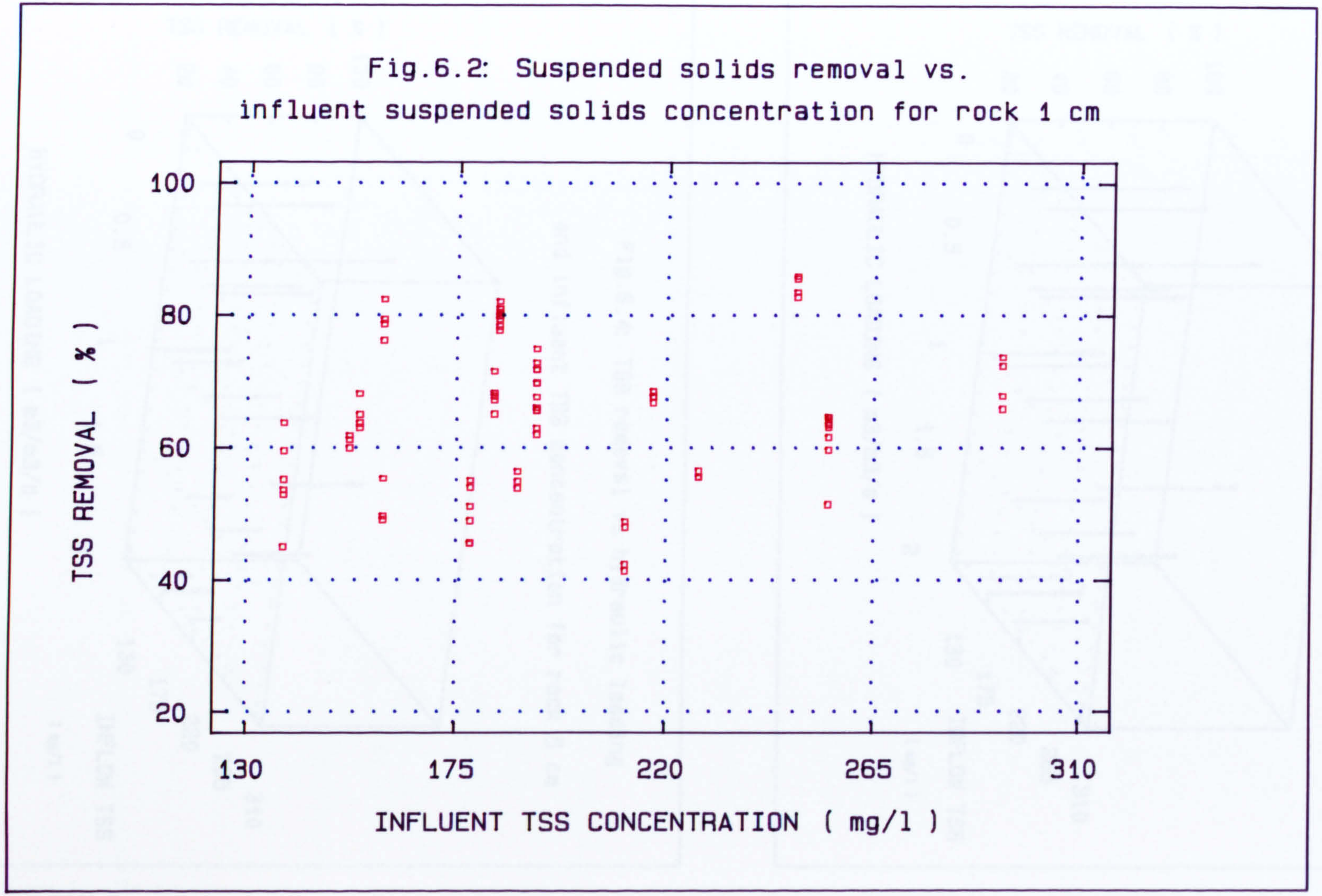


Fig6.3: TSS removal vs. hydraulic loading

and influent TSS concentration for rock 1 cm

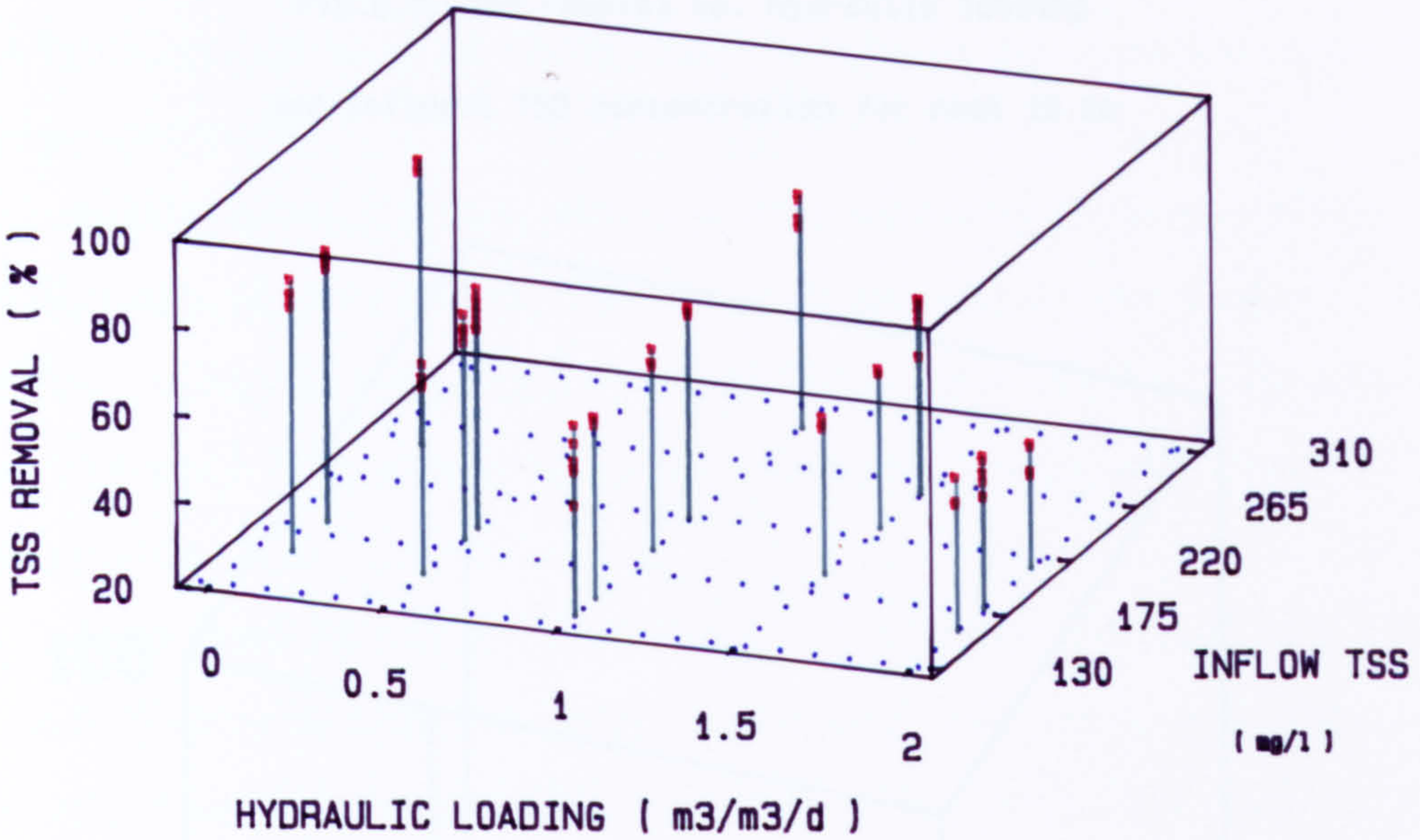


Fig.6.4: TSS removal vs hydraulic loading

and influent TSS concentration for rock 5 cm

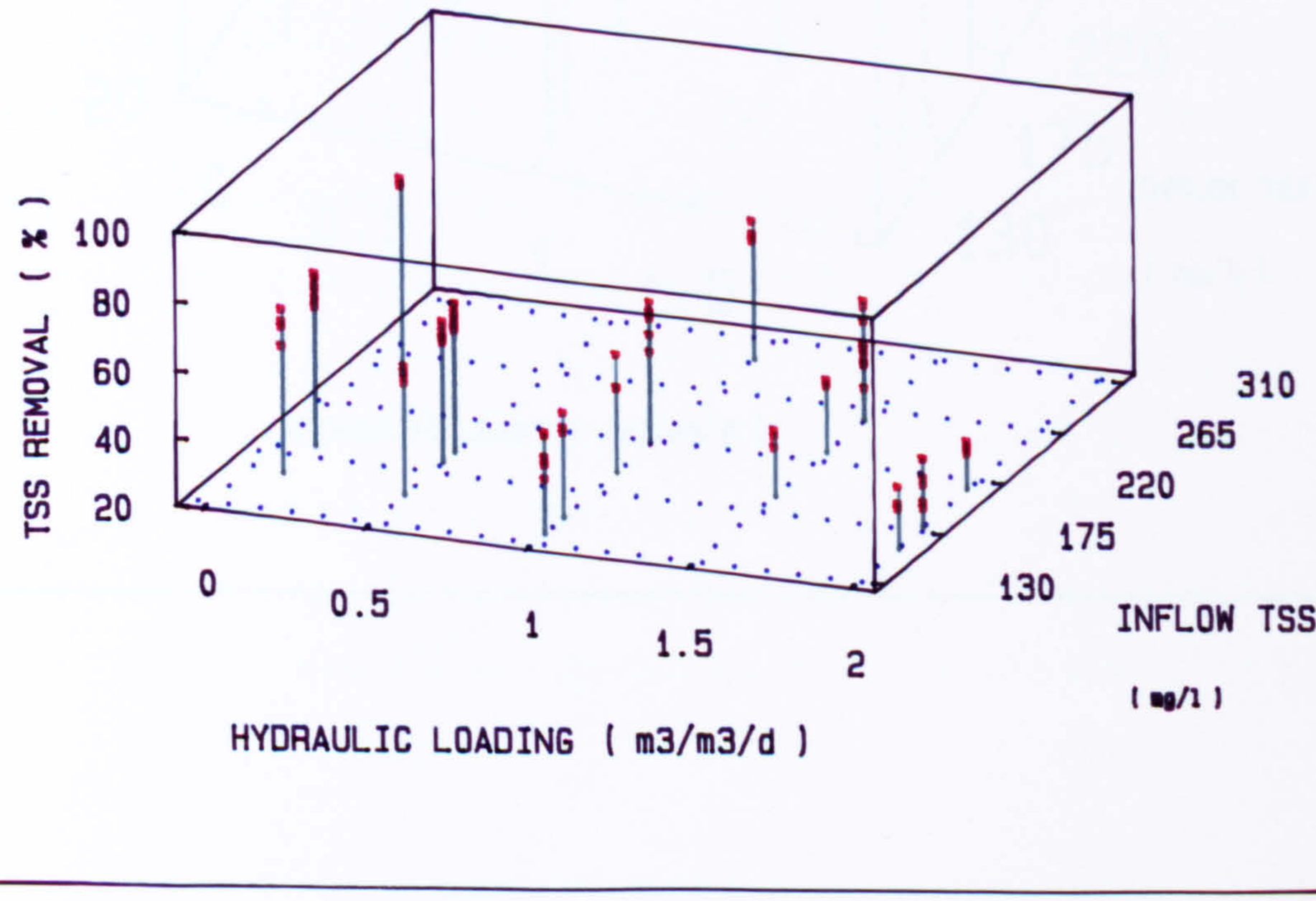
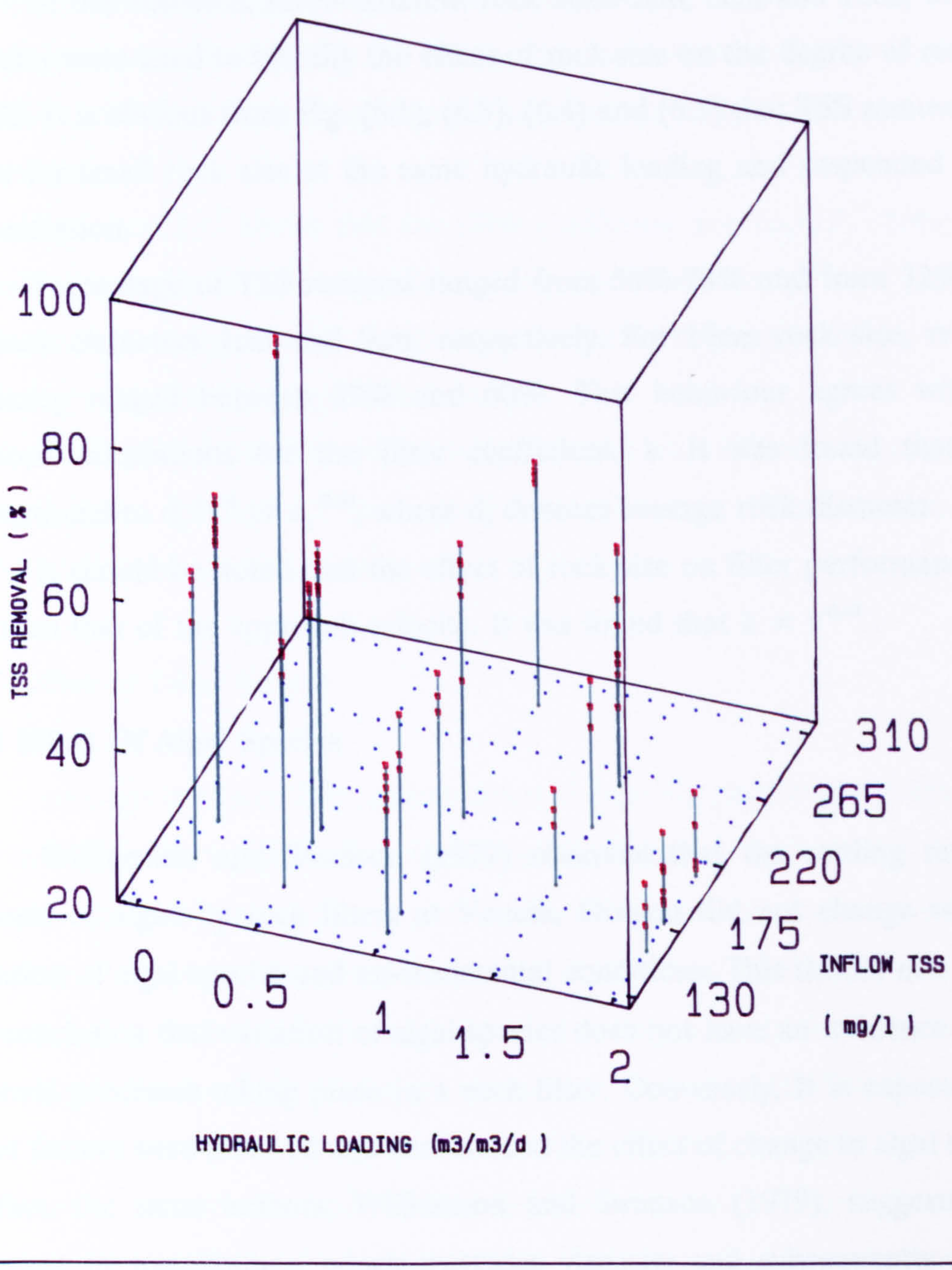


Fig.6.5: TSS removal vs. hydraulic loading

and influent TSS concentration for rock 10 cm



TSS is affected by rock filter size. It was recommended by O'Brien (1975,1976) that hydraulic loading of 1203 l/m³.d should be decreased if rock size larger than 2.54-12.7 cm is used. He did not indicate how much this decrease should be or how it is calculated. In fact, O'Brien and McKinney (1979) found that both large (0.95-5.08 cm) and small (0.47-2.54 cm) rock filters removed the same quantity of suspended solids.

In this research, three different rock sizes 1cm, 5cm and 10cm average diameter were used to identify the effect of rock size on the degree of removal of TSS. It is obvious from Fig. (6.1), (6.3), (6.4) and (6.5) that TSS removal was better for small rock size at the same hydraulic loading and suspended solids concentration.

Percentage of TSS removal ranged from 50%-78% and from 32%-67% for rock diameters 1cm and 5cm, respectively. For 10cm rock size, removal efficiency ranged between 27% and 60%. This behaviour agrees with the previous calculations for the filter coefficient, λ. It was found that λ is proportional to $d_c^{-\alpha-\beta-\delta}$ or $d_c^{-0.30}$, where d_c denotes average rock diameter.

It should be noted that the effect of rock size on filter performance was less than that of the approach velocity. It was found that $\lambda \propto v^{-0.40}$.

6.2.4 Effect Of Algal Species

Williamson and Swanson (1979) observed that the settling rate and removal of algae by rock filters at Veneta, Oregon did not change with the variation of algal species and environmental conditions. This should not lead to the conclusion that variation of algal species does not have an influence on the removal processes taking place in a rock filter. Conversely, It is expected that other factors were involved and confounded the effect of change in algal species. In fact, the same authors, Williamson and Swanson (1979), suggested that decrease in temperature, which increases viscosity and subsequently hinders settling, is accompanied by the appearance of species that sink faster.

Many researchers investigated the sinking rates of algae and found different types of algae settle at different velocities (Eppley et al., 1967; Bella, 1970; Smayda, 1970; Titman and Kilham, 1976). This agrees with the assumption of Ramani (1976) that filamentous algae may be removed more easily than non-filamentous ones.

In Section 2 of this Chapter it was found that settling and hydrodynamic action are the most significant mechanisms in algae removal. Both of these mechanisms are affected by the shape, size, and density of the algal cells. Therefore, different algal species could be expected to have different removal efficiencies.

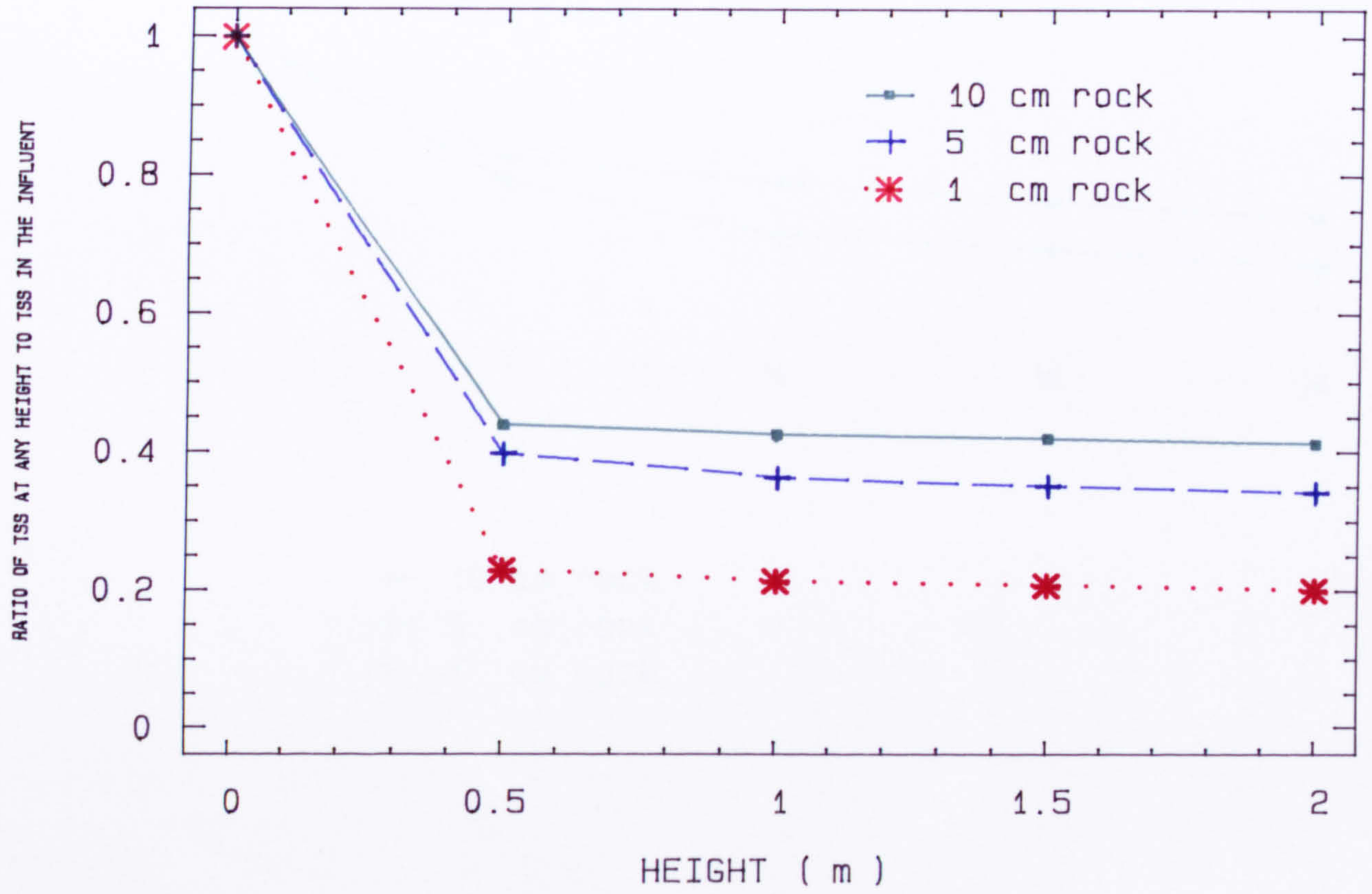
Equation 2.11 shows that the filter coefficient $\lambda \alpha (\rho_s-\rho)^{\gamma}$. Solution of this equation in Section (6.1.2) yielded λ is proportional to $d_p^{0.58}$ and $(\rho_s-\rho)^{0.226}$, which means that algae removal efficiency is affected by particle diameter but to a less degree by change in particle density.

Table (6.2) shows that different algal species with different sizes appeared in the influent throughout the experiments. However, *Chlamydomonas and Euglena* were the most dominant.

6.2.5 Effect of Filter Height

Average values of TSS concentration at different heights of the filter were introduced in Table (5.3). For hydraulic loadings 0.1 and 2m^3/m^3.d, as an example, Fig. (6.6) and (6.7) are plotted. These Figures reveal that most of the TSS removal was accomplished within the first 50 cm of the filter and little removal was contributed by the rest of the filter height, for both the low and the high hydraulic loadings. This phenomenon may be attributed to some physiological changes that algae undergo when entering the filter. Under dark conditions, algae are exposed to stress and start producing exocellular enzymes, which cause autoflocculation (see Section 6.1.3 -II). Due to autoflocculation enhanced by orthokinetic flocculation, algal flocs are formed and removed mostly in the first part of the filter.

Fig.6.6: Variation of TSS concentration with height of the filter for hydraulic loading of 0.1 m3/m3.d



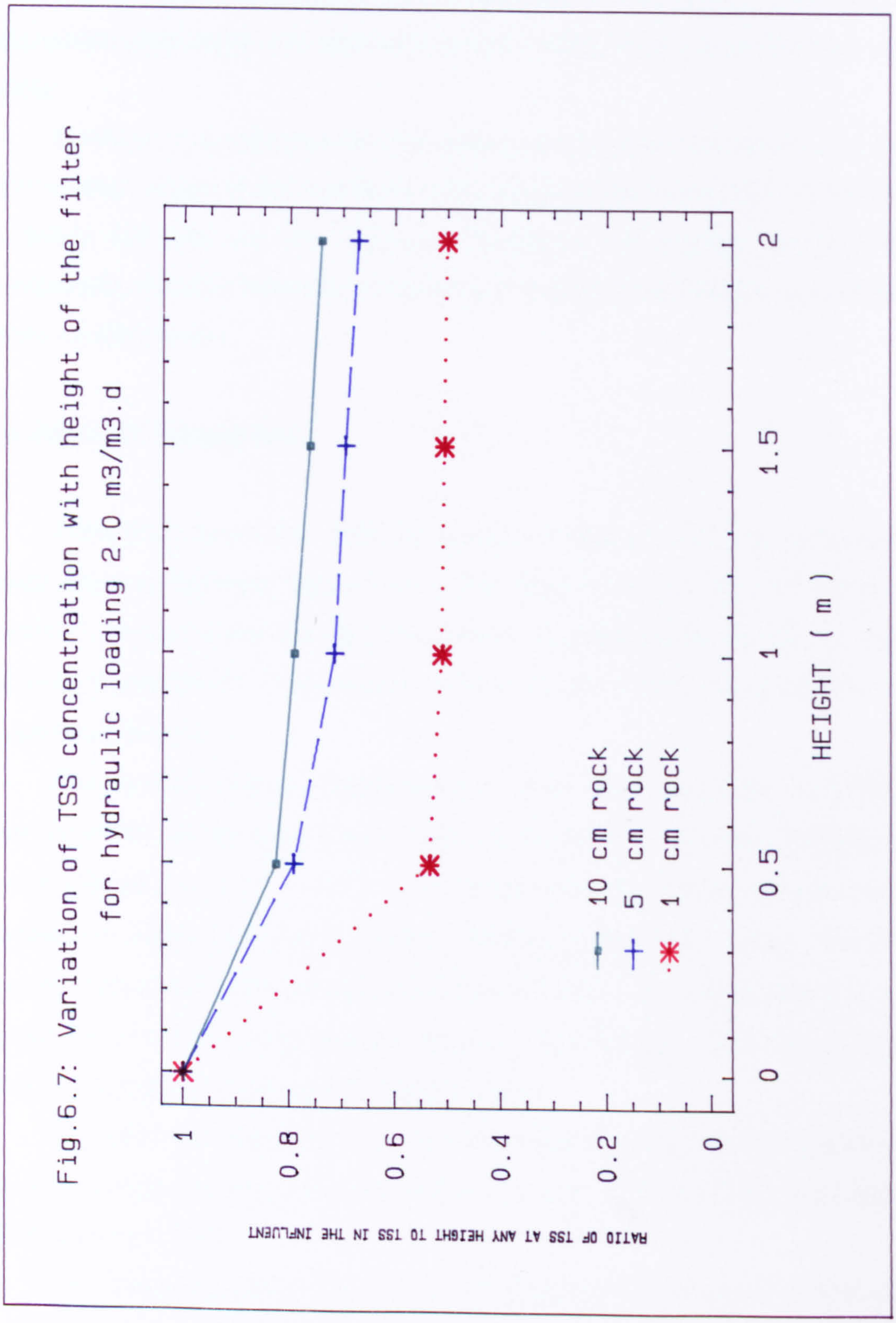


Fig.6.7: Variation of TSS concentration with height of the filter
for hydraulic loading 2.0 m3/m3.d
RATIO OF TSS AT ANY HEIGHT TO TSS IN THE INFLUENT
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
HEIGHT (m)
10 cm rock
5 cm rock
1 cm rock

Thus, it can be concluded that it is not favourable to design rock filters using volumetric hydraulic loading, as has been practised until now (see, for example, O'Brien and Mckinney, 1979). This would overestimate the performance of the filter because only part of the filter volume is actually effective in treatment at a certain time. Therefore, it is desirable to relate filter performance with depth and approach velocity rather than volumetric hydraulic loading.

However, it is expected that the upper parts of the filter would come into effect in later stages of the treatment when pores in the lower part of the filter are nearly full and are less effective. Therefore, the storage capacity and, consequently, the time between backwashes, is increased and based on the whole volume of filter pores.

6.2.6 Effect Of Temperature

Table (6.2) shows that different species of algae prevailed in the influent to rock filters at different temperatures. The effect of temperature in this sense is indirect; it plays a role among other factors in determining the type of algae that exist in abundance. Consequently, removal will be affected, as explained in the previous Section.

The second effect of temperature is direct. It changes the viscosity of water. However, in this work it was earlier shown that the removal coefficient λ is proportional to $\mu^{-\beta-\gamma+\delta} = \mu^{-0.05}$. This reveals that the effect of variation of viscosity on algae removal is minute. Raising temperature from 9°C-34°C decreases viscosity from 1.307×10^{-3} to 0.74×10^{-3} N.S/m^2. This means that λ is only affected by a factor of 1.03 because there are mechanisms, other than settling, which are acting in the process of algae removal.

The effect of temperature on viscosity cannot be used alone to explain the variation of algae removal with temperature (Smayda, 1974 and Stutz-McDonald and Williamson, 1979).

The balancing effect observed by Williamson and Swanson (1979), who showed that lower viscosity is offset by the appearance of species with faster

settling rates, may not be valid. In fact, the above authors reached this conclusion because they assumed that settling is the main mechanism of algae removal.

In this research, it has been explained that other mechanisms, in addition to settling, have an effect on the process of removal of algae in rock filters and result in viscosity change due to temperature change having only a minute effect. In fact, the results of the above researchers (Williamson and Swanson, 1979) may be explained in terms of TSS concentration and hydraulic loading. It is noticed that species of lower settling rates (blue green algae) appeared in higher concentrations in Summer when hydraulic loading was low, which increased the percentage of removal. Not only that, these species had smaller diameters, which enhance their removal by other mechanisms such as diffusion and hydrodynamic forces. Change of viscosity due to change in temperature is not sufficient explanation of the high percentage of removal of blue green algae, which have a low settling rate because of their small diameters and low cell density.

6.2.7 Interaction Between Hydraulic Loading and TSS

Bernardo (1988) found that the increase in effluent turbidity due to increase in influent turbidity, was independent of approach velocity. On the contrary, the results of this research show clearly that effluent quality is affected by influent quality to different degrees depending on the approach velocity (see Section 6.2.2). To identify further the interaction between the effect of hydraulic loading and influent TSS concentration for different rock sizes, the experiments were carried out according to composite design requirements introduced in Chapter 4. To solve the full second order model (Eq. 4.1) each factor should be taken at five levels to produce nine combinations.

$$Y_{1i} = \beta_{\bullet} + \beta_1 x_{1i} + \beta_2 x_{2i} + \beta_{11} x_{1i}^2 + \beta_{22} x_{2i}^2 + \beta_{12} x_{1i} x_{2i} + r_{1i} \quad \ldots\ldots (4.1)$$

Since it was not possible to hold TSS at predetermined levels, the nine combinations of the two factors would only be possible if a range of a TSS concentration is considered to represent an average level, which may not be accurate enough. However, looking at the average influent TSS (Table 5.2 and Table 6.3) shows that seventeen levels were tested. Three of the levels tested were repeated, which leaves fourteen different levels of TSS concentration. Instead of the nine minimum combinations required, fourteen combinations in addition to three replicates can be employed for solving Eq. 4.1.

Data from Table (6.3) were used to solve Eq. 4.1, with Y_1 representing average TSS removal, x_1 hydraulic loading and x_2 influent TSS average concentration. The estimated parameters for Eq. 4.1 are tabulated below in Table (6.5).

Table 6.5: Estimated parameters for equation 4.1

PARAMETER	ROCK 10 cm	ROCK 5 cm	ROCK 1 cm
$\beta_.$	27.133	32.885	57.335
β_1	-4.570	-6.397	-14.449
β_2	0.204	0.224	0.133
β_{11}	0.384	1.961	3.193
β_{22}	-0.000087	-0.0002	-0.00003
β_{12}	-0.078	-0.0086	-0.042

For the filters of 10 cm and 5 cm size β_{12} was found significant at levels 0.002 and 0.07, respectively. For 1cm rock, β_{12} was found insignificant with a significance level of 0.31.

However, the interaction between hydraulic loading and concentration of suspended solids in the influent is still noticeable for the three filters in Fig. (6.8), (6.9) and (6.10). Generally, TSS removal increased with increase of TSS concentration in the influent, especially for low hydraulic loading. For high hydraulic loading, a small improvement in TSS percentage removal occurred when influent TSS concentration increased at that loading. For example, when the filter with rock size of 10 cm was operated at a hydraulic loading of 0.5 $m^3/m^3.d$, increase of influent TSS from 160 to 240 mg/l increased TSS removal from 50% to 60%. For the same filter, receiving a hydraulic loading of 1.5 $m^3/m^3.d$, TSS removal improved from 33% to 37% when influent TSS concentration went up from about 160 mg/l to 240 mg/l. The same trend is applicable to the filter with 1 cm average diameter rock.

For the filter with 5 cm rock size, the effect of TSS concentration in the influent, on TSS percentage removal, is almost equal at all tested hydraulic loadings. For example, at a hydraulic loading of 0.5 $m^3/m^3.d$ and an increase of influent TSS from 160 mg/l to 240 mg/l, the percentage removal of TSS changed from 60% to 71%. For the same change of influent total suspended solids concentration, the percentage removal of TSS increased from 56% to 67% at 1.5 $m^3/m^3.d$ hydraulic loading.

Such behaviour may be attributed to the dominant mechanism of removal in each filter. Pore size has an important role in this respect. In the small rock filter, hydrodynamic action may be dominant over settling, while the reverse may be occurring in the filter with 10 cm rock. This suggests that the performance of the filter becomes sensitive to change in velocity more than change in influent solids concentration. On the other hand, for the medium size filter media the two mechanisms become equally important and support each other such that the system becomes less sensitive to velocity change and gives way for other factors to affect its performance.

Fig.6.8: Suspended solids removal
equation (4.1) for rock 1.0 cm

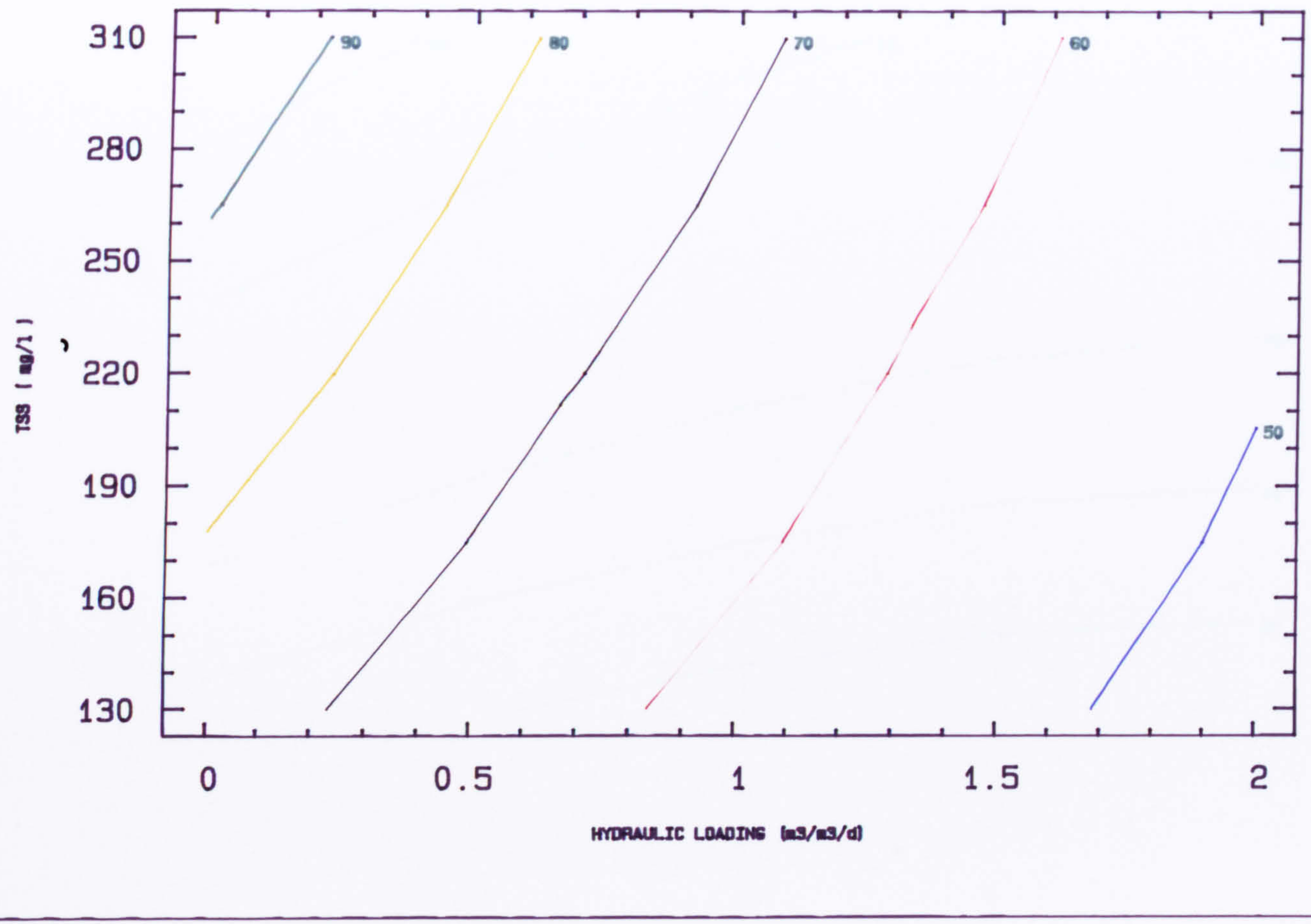


Fig6.9: Suspended solids removal
equation (4.1) for rock 5 cm

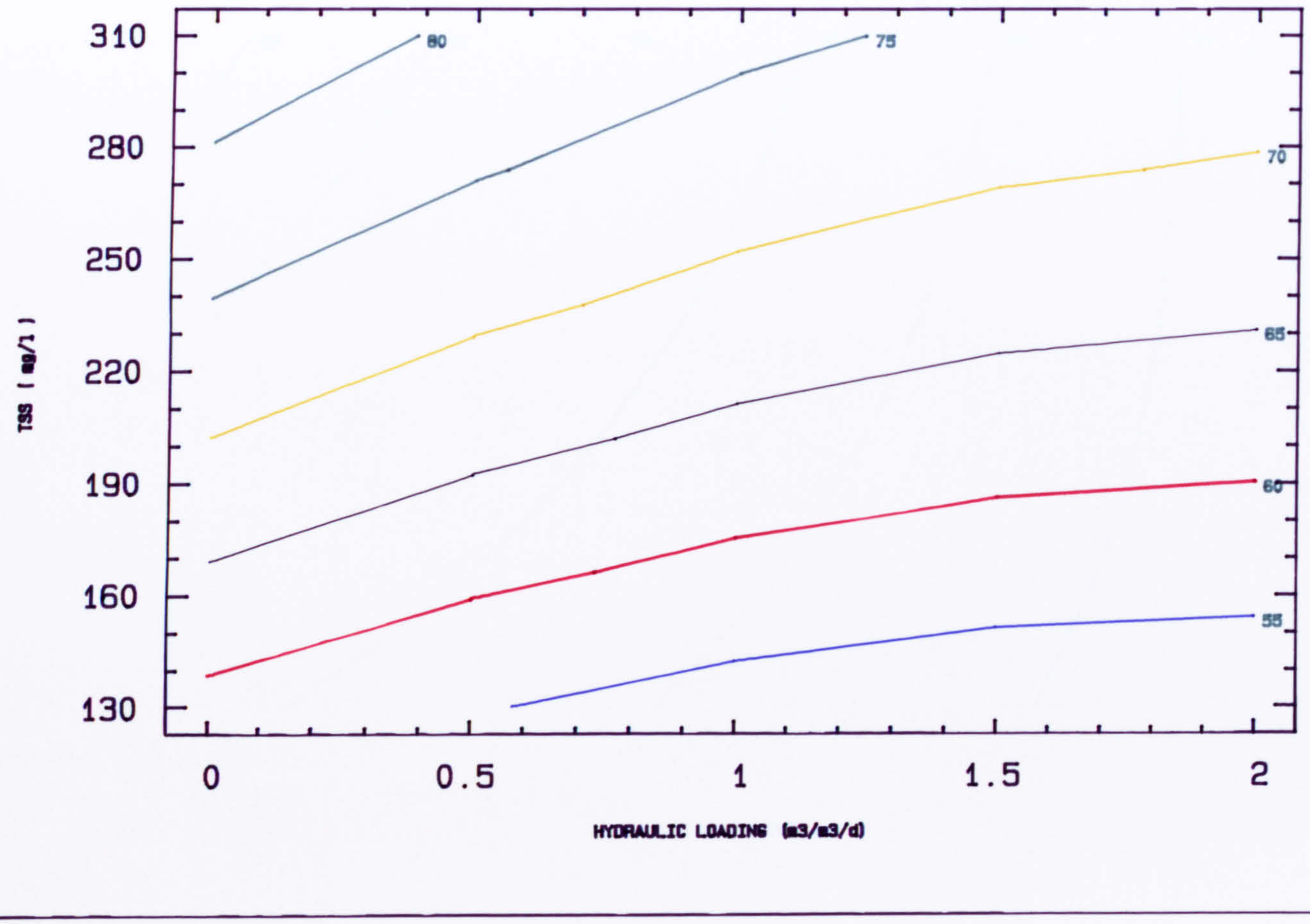
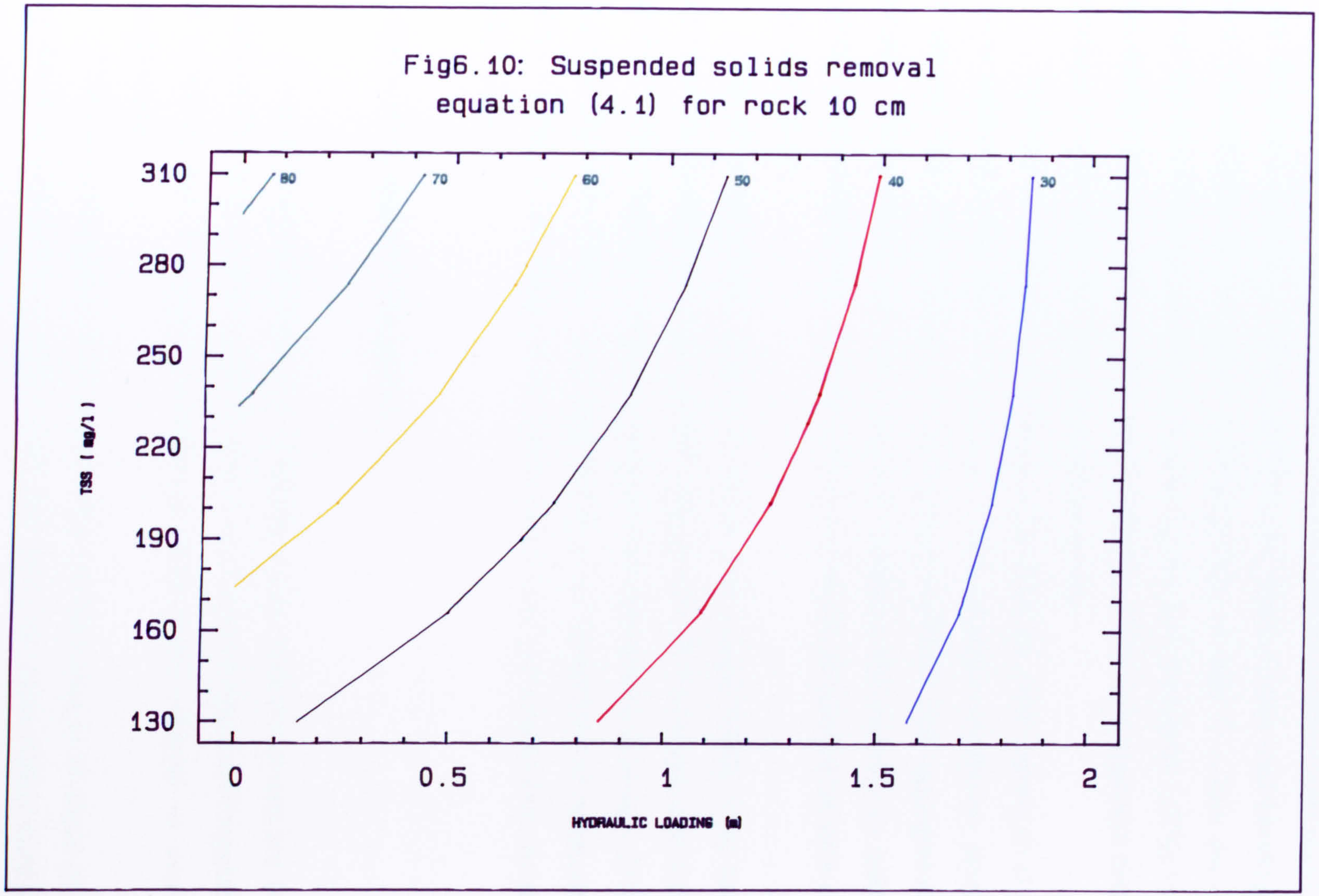


Fig6.10: Suspended solids removal
equation (4.1) for rock 10 cm



Another explanation may relate to the degree of flocculation in each filter. The effect of turbulence, represented by G, during the flash mixing and flocculation periods in the jar test or field scale units has been discussed by many researchers (e.g. Camp, 1955, 1968; Stevenson, 1980). Boadway (1978) and Huck and Murphy (1978) noticed that high G values enhance collision and growth of flocs. However, too high a velocity gradient will cause erosion and breakage of flocs. Nevertheless, the formation of smaller flocs at high G values does not mean that they are less settleable. In fact, Tambo and Watanabe (1979) found that smaller flocs are denser and more settleable. Thus, it is expected that an optimum G value exists for best flocculation results.

It seems that optimum conditions of flocculation, with respect to G and t, were achieved in the filter of 5 cm rock. Under optimum conditions, increase of influent TSS concentration would promote collision and aggregation of particles. When values of G and t are not optimum, the increase in influent concentration would have a negligible effect on removal, as noticed for rock sizes 1 cm and 10 cm.

Similar results were obtained by Ayoub and Nazzal (1988). They found that a baffled channel flocculator packed with gravel 5.08-7.6 cm (porosity 0.545) had 6.0% better turbidity removal compared with that achieved using 1.27-2.54 cm gravel (porosity 0.478). They suggested that the occurrence of optimum G and/or t values in the pores of the larger gravel size was responsible for such performance.

6.3 Removal of Algae by Coagulation

Coagulation-precipitation in combination with oxidation ponds has been investigated by many workers. Balmér and Vik (1978) used chemical coagulation and sedimentation to remove phosphorus from pond effluent to control eutrophication.

Different compounds have been tested for the removal of algae in the coagulation-sedimentation process. Alum has been the most widely used and successful coagulant (Parker, 1976).

The cost of coagulant is the main factor that restricts the use of the coagulation process in removal of algae from pond effluent. Golueke and Oswald (1965) reported 99% removal of algae at 75 mg/l dose and 100% removal at 90 mg/l dose. Alum was used for post treatment of ponds effluent by Shindala and Stewart (1971) and an optimum dose of 75-100 mg/l was obtained.

In this research, it was found that reduction of alum dose was achieved by increase of settling time or by use of coagulant aids.

The cost of coagulant may be also reduced by employing low cost materials as primary coagulants. Examples of these are kaolinite and bentonite.

6.3.1 Sedimentation Time

The effect of detention time in sedimentation tanks on removal of chemically coagulated algae flocs is not a point of consensus in the literature. Golueke and Oswald (1967) noted that increase of sedimentation time beyond 15 minutes had an insignificant effect on removal of algae by alum coagulation, except when flocculation conditions were less than optimum. A sedimentation time of 2-3 hours was employed in their field-scale experiments.

Speedy et al. (1969) noticed that a short settling time for algae coagulated by alum in water treatment caused the light and small algal flocs to be carried over the effluent weir. They recommended that settling time be increased for better removal.

In a chemically treated wastewater with alum coagulation-sedimentation, further phosphorous removal was achieved when the treated wastewater was delivered to a polishing pond, due to increased detention time over that of the preceding precipitation unit (Balmér and Vik, 1978).

Settling time and temperature were related in their effect on algae removal (Al-Layla and Middlebrooks, 1975). Settling time up to 90 minutes reduced removal at high temperatures of 40°C and 30°C, while no effect of settling time on algae removal by alum coagulation was noticed when temperature was between 25°C and 10°C. It was suggested that higher temperature and prolonged settling time gave the opportunity for released gases

to cause flotation of algae. Flocs formed at lower temperatures were readily settleable and an extended settling time did not cause resuspension or flotation.

Shindala and Stewart (1971) found that increase of flash mixing time of one or two minutes overcame the problem of gases being trapped in the forming flocs. In this work, it was found that sedimentation time affected optimum pH and also influenced the degree of removal and coagulant dose.

(I) Sedimentation Time and Optimum pH

Fig. (5.8a) shows the relationship between residual turbidity in the supernatant and pH for 40 minutes settling when kaolinite was the primary coagulant. Minimum turbidity was obtained in the pH range 2-2.7. Then the level of turbidity started to increase until a pH of 6 was reached, at which the level of turbidity decreased sharply then continued decreasing at a slower rate at higher pH values. A second local minimum of turbidity was obtained at pH 9-10.

For one day and three days settling, an optimum pH was noticed at pH 7-8 (Fig. 5.8b). Increase in settling time shifted optimum pH to lower values. A possible explanation may be related to the type and rate at which different species present in the water are adsorbed onto kaolinite or bonded to its surface. Adsorbed species tend to change optimum pH for coagulation because they change surface charge density and zeta potential and change the mode of interaction among clay unit layers.

Edge to face coagulation for kaolinite occurs when both negative charges on the surface and positive charges are at medium density. This situation takes place at pH 4-5 (Greenland and Mott, 1978). The isoelectric point for edges of kaolinite has been reported to range between 4 and 8 (Dixon, 1977; Greenland and Hayes, 1978; Tan, 1982). This variation of reported isoelectric point for kaolinite may be attributed to adsorption of charged molecules in a similar fashion to what has been noticed on positively charged aluminum hydroxide. The isoelectric point decreased upon adsorption of sulphate anions and was driven to higher levels due to addition of calcium (Hsu, 1975; Letterman and Sricharoenchaikit, 1982). Adsorption of sulphates and chlorites shifted the pH

of alum towards the acidic side because more H^+ is needed to counterbalance the new negative charge (Bratby, 1980).

Letterman and Vanderbrook (1983) suggested that sulphates anions will compete with negative clay particles for adsorption on positively-charged hydrolysed aluminium compounds. This requires higher doses of alum coagulant, and lower pH values to shift electrophoretic mobility from negative to positive values. Narkis and Rebhun (1977) noticed similar behaviour with cationic polyelectrolytes. Adsorption of organic matter (anionic polyelectrolytes) gave a lower optimum pH for coagulation.

Reference to Fig. (5.8a), together with the knowledge that kaolinite edges and basal planes become positively charged at pH<3 (Greenland and Hayes, 1978), may explain why an optimum was obtained at pH2-2.7 for 40 minutes settling. In this pH range, negatively charged algae are bonded to positively charged kaolinite. Above pH 3, the surface is not amphoteric and increase of pH increases its negative charge density and zeta potential due to adsorption of OH^- by the silanol group, $Si\text{-}O^-$. This may explain the increase of turbidity when pH was increased above 3. However, further increase in pH causes the surface charge of algae and its zeta potential to become minimum (pH 7-8, Ives, 1956, 1959), and kaolinite edges approach their isoelectric point, which may explain why coagulation took place again.

As stated earlier, the change of optimum pH to lower values (Fig. 5.8b), when settling time was increased, may be related to adsorption of some negative species onto kaolinite and/or algae surfaces. It seems that the adsorption of some anions on the surface of algae and kaolinite takes a longer time than reactions associated with H^+ and OH^- and metal cations. The reactions are instantaneous but the rate of diffusion of these anions in the double layer may be slow. These anions shift the isoelectric point towards lower levels of pH. Raising the negative surface charge density and zeta potential of kaolinite requires that coagulation be more effective at lower pH values, in order to overcome repulsive forces and decrease the energy barrier.

However, this decrease of optimum pH was not continuous because: first, there is a limit to the amount of anions that could be adsorbed according to the

availability of adsorption active sites; second, positive species such as Ca^{++} and Mg^{++} may also be adsorbed on the clay surface and pull the isoelectric point upwards.

Fig. (5.9) shows that with bentonite also the optimum pH shifted from 9 to 7.2 when settling time increased from 40 minutes to 3 days. This behaviour may be similar to that of kaolinite .

Shift of optimum pH near neutrality has an economic advantage in that there is no need for post treatment by other chemicals in order to lower or raise pH to environmentally acceptable levels. If effluent is used for irrigation, then the pH range required is 6.5 to 8.4 (Pescod, 1992), which coincides with optimal pH values, obtained in this research, for coagulation by kaolinite and bentonite at longer settling times.

(II) Sedimentation Time and Dose of Alum

In Fig. (5.10) it was shown that optimum alum dose for 40 minutes settling was about 450 mg/l. This figure is very much higher than the optimum dose of 105-120 mg/l reported by Golueke and Oswald (1965); also higher than 250-300 mg/l alum for algae removal reported by Ford and Tischler (1976). The reason may be due to high alkalinity of the samples tested in this work (770 mg/l-$CaCO_3$). Low turbidity water with high concentration of alkalinity requires excessive alum doses for effective coagulation by sweep floc (O'Melia, 1972). Another possibility could be the presence of sulphates. Letterman and Vanderbrook (1983) found that the presence of sulphate raised the dose of alum required for effective coagulation of clay suspension. The concentration of sulphates, from March, 1990 to March, 1991, in the effluent of Al-Samra Wastewater Stabilization Ponds ranged between 3.8 and 45 mg/l (RSS, 1991)

In this work, tests were carried out to investigate the possibility of decreasing alum dose by increasing sedimentation time. Fig. (5.11) shows that longer sedimentation time gives the opportunity for more solids to settle at doses of alum up to 80 mg/l, although the effect was more evident when the dose was less than 40 mg/l. Fig. (5.11) reveals the type of flocs that were formed with

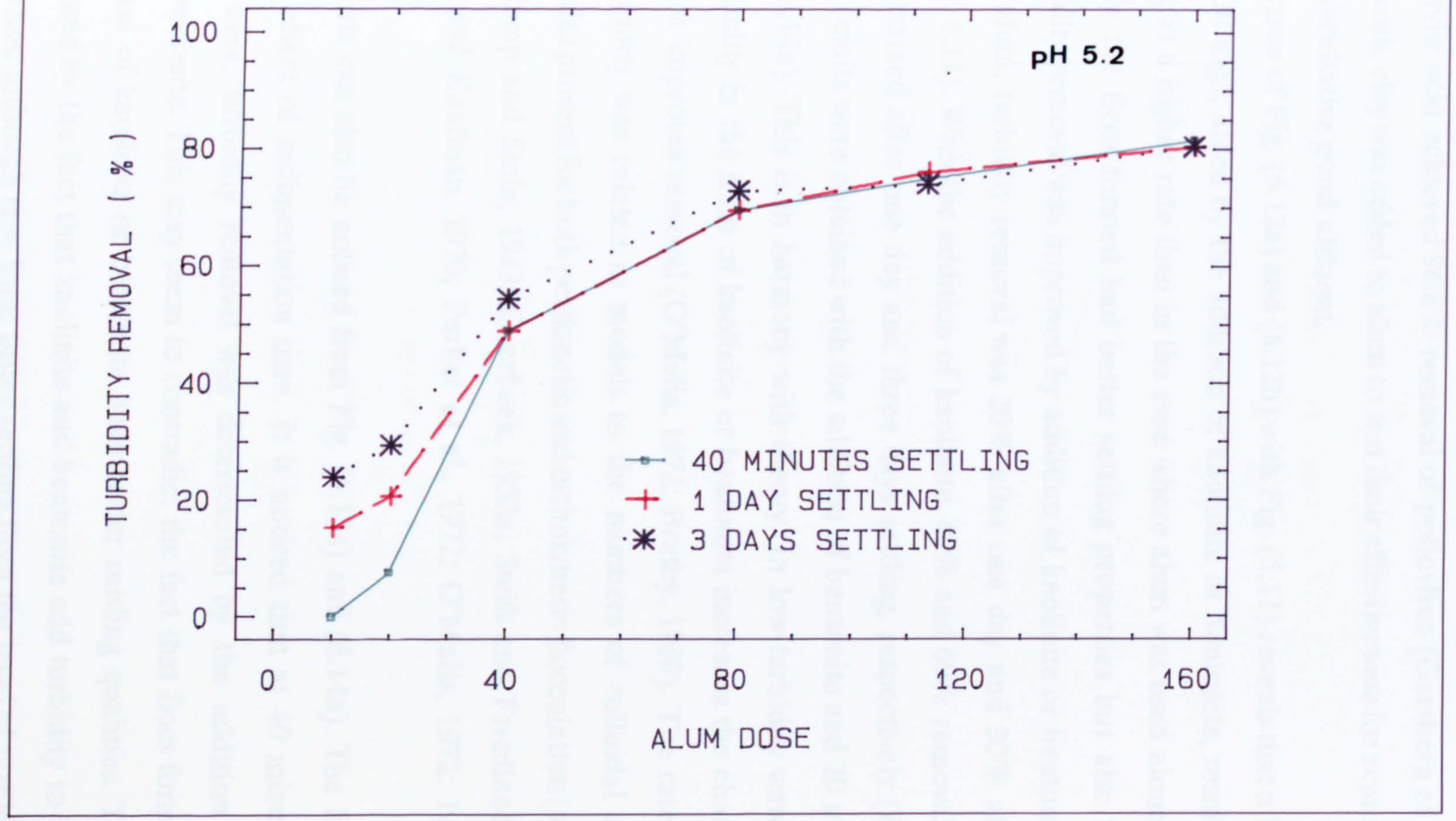
different alum doses. At 10 and 20 mg/l, small uniform and less dense flocs, which settle slowly, were present. At higher doses of alum, highly settleable flocs were dominant. The latter flocs settle fast and little improvement was attained with time, by settling of remaining smaller, less dense flocs which only form a small proportion of the total number of the formed flocs.

Removal of turbidity at different settling times was calculated with respect to the turbidity of the raw wastewater. The results are shown in Fig. (6.11). When the dose was more than 40 mg/l there was no difference between turbidity removal at 40 minutes and 1 day. However, after three days of settling, further turbidity removal of about 5% took place for doses of 40-80 mg/l. For doses of 20 mg/l, 20% more turbidity removal was achieved after 3 days than was obtained after 40 minutes. The total turbidity removal at 20 mg/l alum and three days settling was about 30%.

The fact that at certain doses of coagulant the time of sedimentation affects turbidity removal, and at other doses all results converge to the same removal value, suggests that the settleability of formed floc is affected by coagulant dose. It also shows the importance of carrying out multiple sampling jar tests rather than taking results at a fixed point in time, especially for design purposes. Such results agree with those obtained by Reed and Robinson (1984).

The relationship shown here (Fig. 6.11) between settling time and coagulant dose with turbidity removal may explain inconsistencies among researchers with regard to the effect of settling time on clarification. It is clear, when the dose is optimum or near optimum, good settleable flocs will form and will be removed at a high rate from suspension. On the other hand, flocs that are formed with doses far from optimum will settle slowly and the effect of settling time will be obvious. This conclusion is in agreement with that of Golueke and Oswald (1967), who stated that sedimentation time in the jar test beyond 15 minutes had an insignificant effect on removal of algae, except when flocculation conditions were less than optimum.

Fig.6.11: Removal of turbidity vs alum doses
after different settling times



6.3.2 Use of Coagulant Aids

Addition of 600-800 mg/l of montmorillonite to 150 mg/l of alum and 1 mg/l of polyacrylic acid achieved 96.8% removal of poliovirus (Gersberg et al., 1988). In this work, clay was added to alum to test their effectiveness for removal of algae from facultative pond effluent.

Comparison of Fig. (6.12a) and (6.12b) with Fig. (5.11) reveals that a low dose of alum, 20 mg/l, aided by the addition of kaolinite or bentonite, resulted in flocs settling at a higher rate than in the case where alum was used alone.

Not only the flocs formed had better settling properties but also the degree of turbidity removal was improved by addition of kaolinite or bentonite. At 20 mg/l of alum, turbidity removal was 20% after one day and 30% after three days (Fig. 6.11). With the addition of kaolinite, 40% and 60% removal of turbidity was attained after one day and three days settling, respectively (Fig. 6.13a). Similar results were obtained with the addition of bentonite and 20 mg/l of alum (Fig. 6.14a). This is in harmony with theory. In low-turbidity waters, addition of turbidity in the form of kaolinite or bentonite increases the chance of contact, which improves removal (O'Melia, 1972; Bratby, 1980). The rate of removal of turbidity was related in models to the numbers of colloidal and coagulant particles present for both perikinetic and orthokinetic flocculation (see, for example, Camp and Stein, 1943; Overbeek, 1952a; Swift and Friedlander, 1964; Argman and Kaufman, 1970; Parker et al., 1972; O'Melia, 1972; Ives, 1978).

Two points can also be noticed from Fig. (6.13a) and (6.14a). The first deals with the effect of sedimentation time. It is noticed that at 40 minutes sedimentation time, turbidity removal was deteriorated by the addition of kaolinite and bentonite. This may seem to contradict the fact that flocs formed with the addition of kaolinite or bentonite have better settling qualities. This could be explained by the fact that kaolinite and bentonite add turbidity to the treated water. Thus, although they form faster settling flocs the level of turbidity after a few minutes of settling may still be higher than the original turbidity of the treated water, which explains the attainment of negative values of turbidity

Fig.612a: Effect of kaolinite on settling vs time
with 20 mg/l of alum

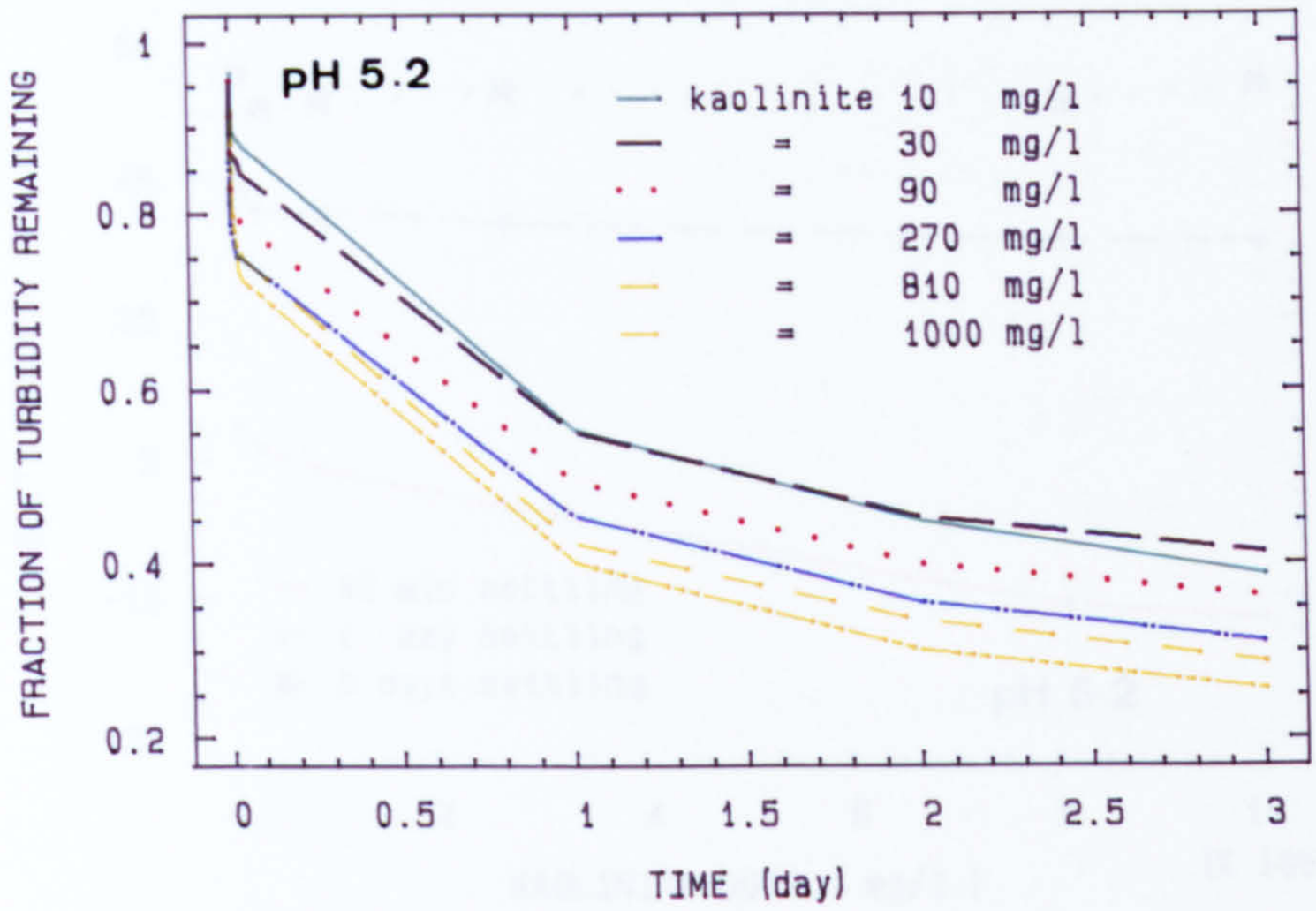


Fig.6.12b: Effect of bentonite on settling vs time
with 20 mg/l of alum

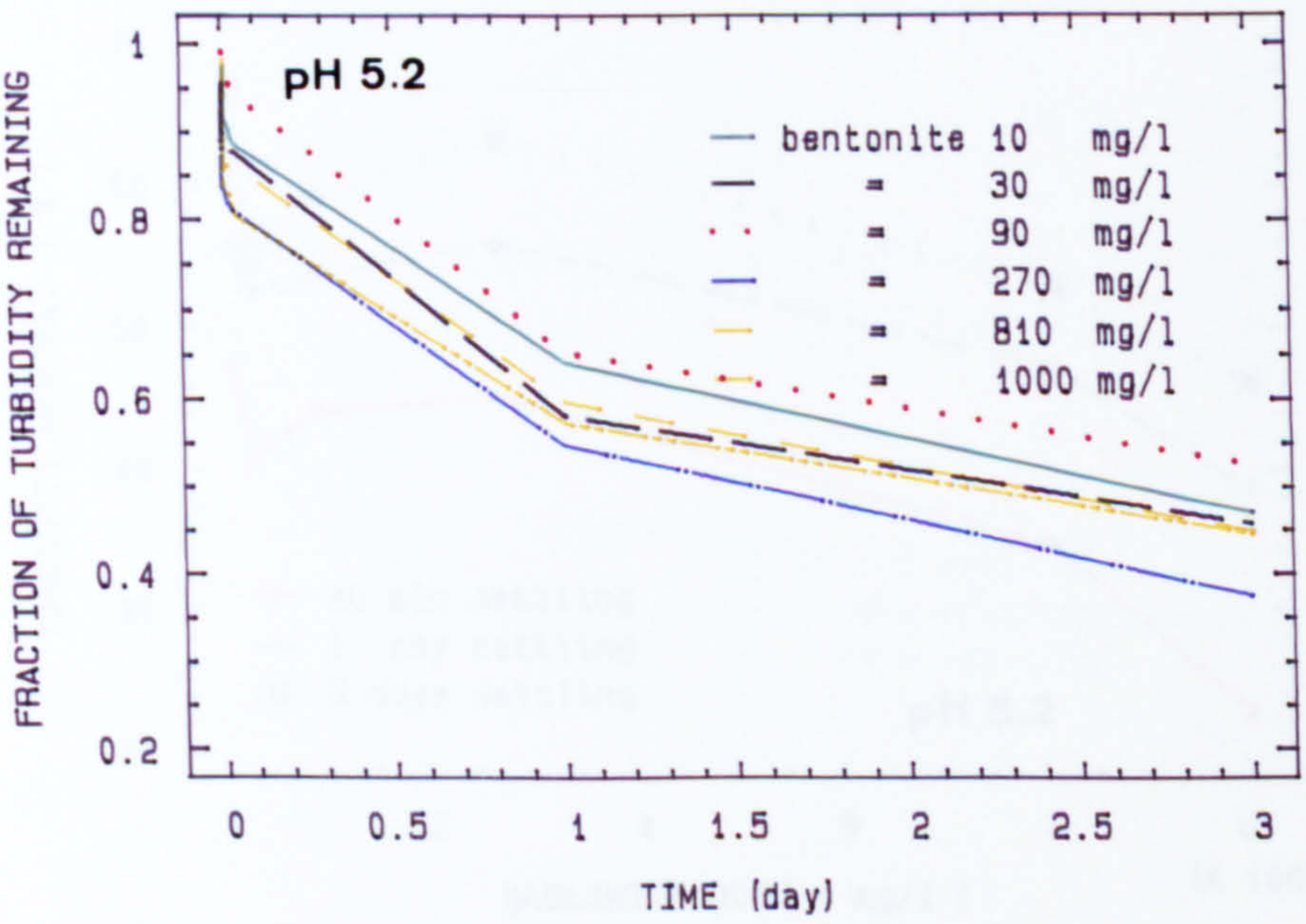


Fig.6.13a: Turbidity removal by 20 mg/l alum
and different kaolinite doses

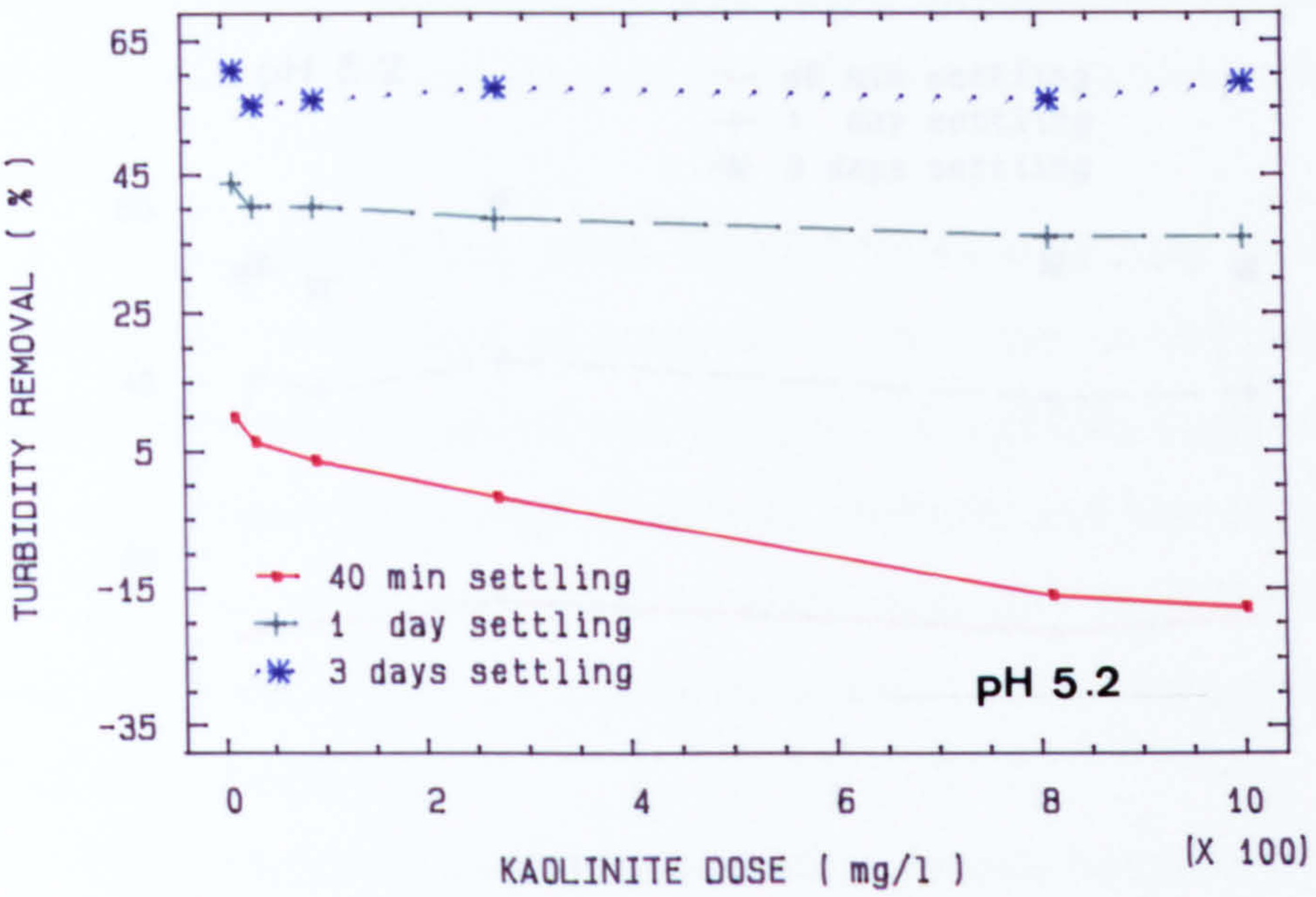


Fig6.13b: Turbidity removcal by 80 mg/l alum
and different kaolinite doses

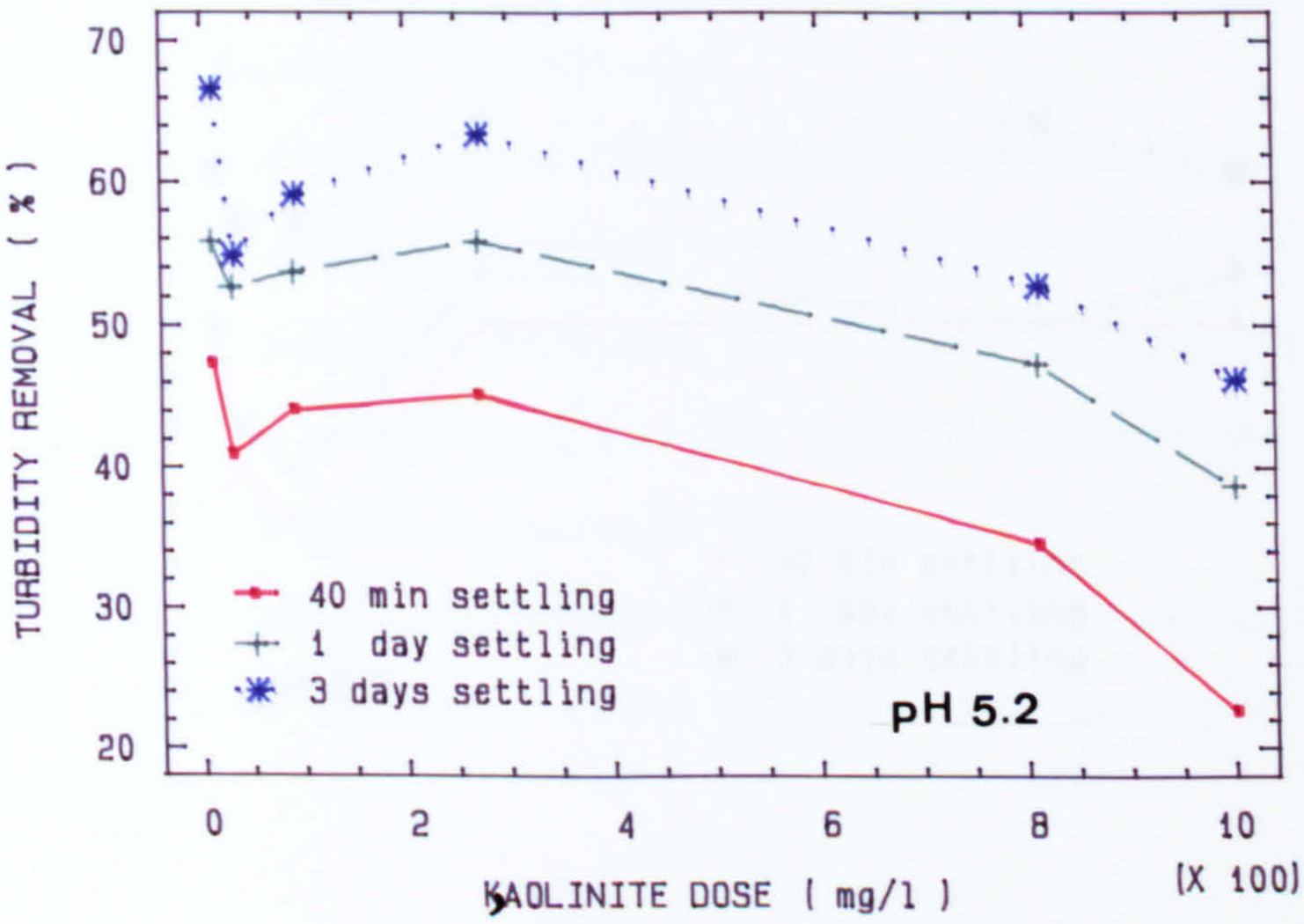


Fig.6.14a: Turbidity removal by 20 mg/l alum
and different bentonite doses

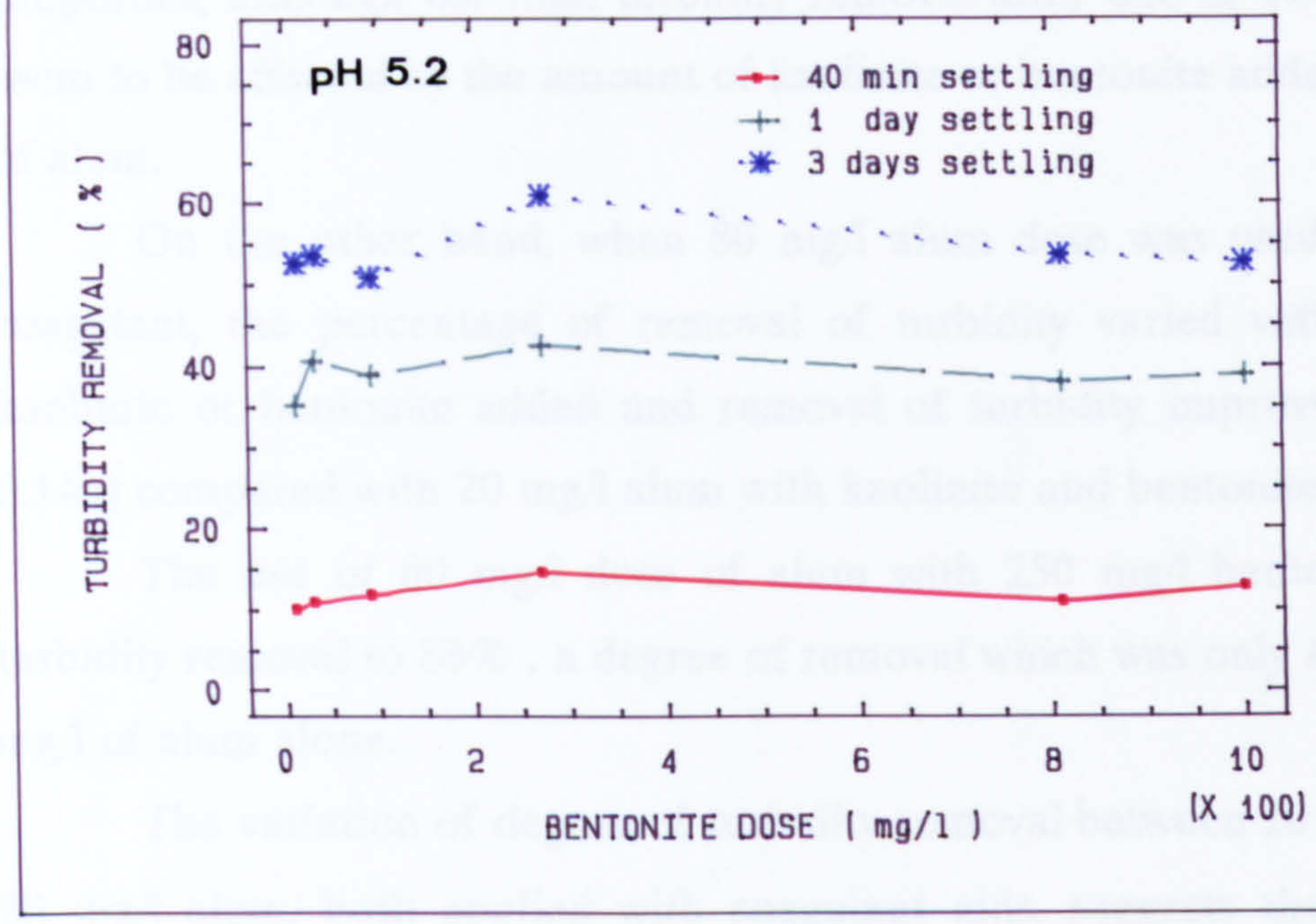
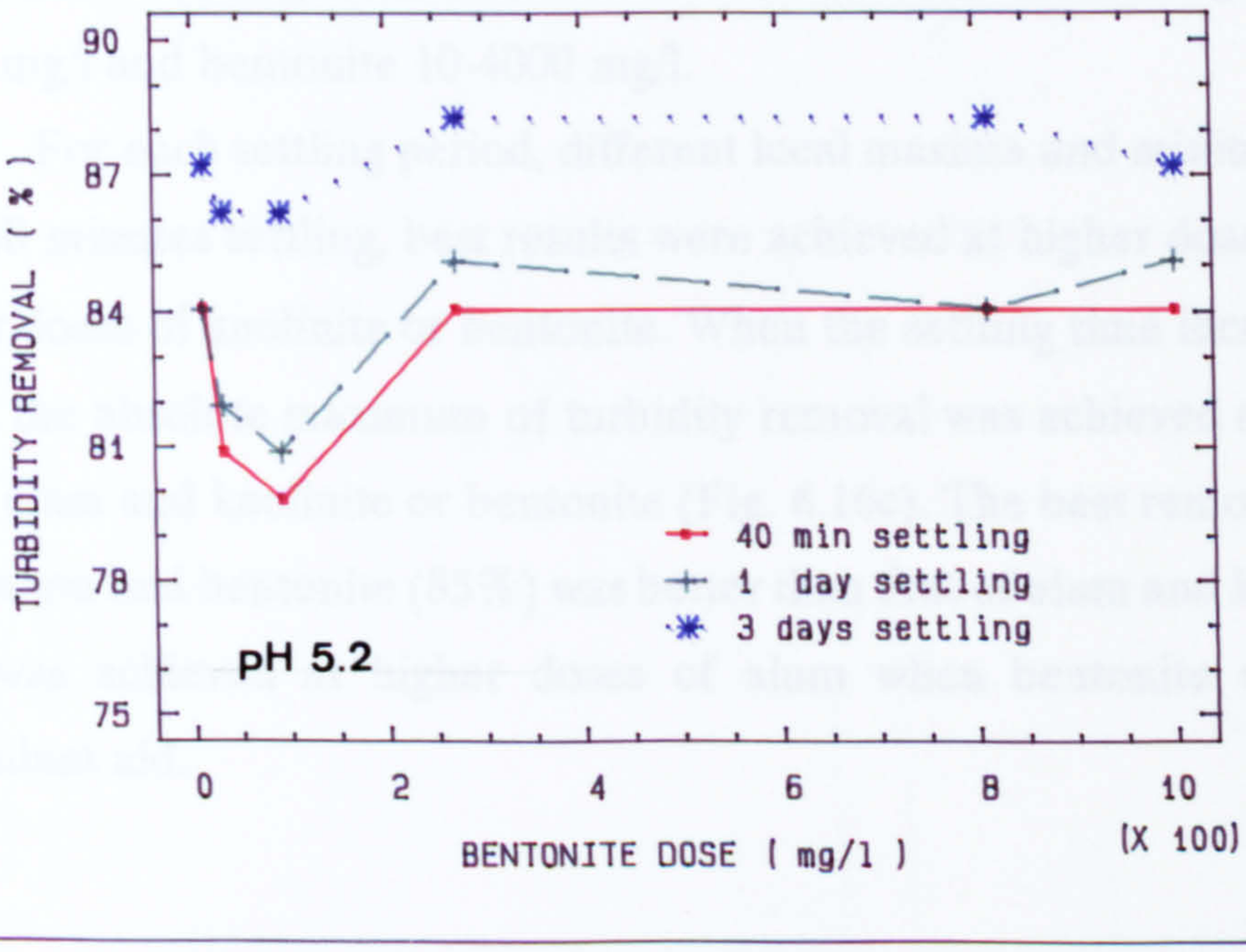


Fig.6.14b: Turbidity removal by 80 mg/l alum
and different bentonite doses



removal after 40 minutes. Given enough time to settle, flocs formed by the addition of kaolinite eventually gave better results than alum used alone.

The second point is related to the effect of a kaolinite or bentonite dose. It is noted that the higher the coagulant aid dose, the better was the settling properties, although the final turbidity removal after one or two days did not seem to be affected by the amount of kaolinite or bentonite added with 20 mg/l of alum.

On the other hand, when 80 mg/l alum dose was used as a primary coagulant, the percentage of removal of turbidity varied with the dose of kaolinite or bentonite added and removal of turbidity improved (Fig. 6.13b, 6.14b) compared with 20 mg/l alum with kaolinite and bentonite.

The use of 80 mg/l dose of alum with 250 mg/l bentonite increased turbidity removal to 88% , a degree of removal which was only achieved at 160 mg/l of alum alone.

The variation of degree of turbidity removal between 20 mg/l alum and 80 mg/l alum, both applied with coagulant aids, suggests that an optimum combination of alum and kaolinite or bentonite must exist for best removal of turbidity.

To substantiate this, different combinations of alum and kaolinite or bentonite were used in jar tests. The results are shown in Fig. (6.15) and (6.16) for different settling times with alum doses between 5 and 80 mg/l, kaolinite 10-1500 mg/l and bentonite 10-4000 mg/l.

For each settling period, different local maxima and minima are noticed. For 40 minutes settling, best results were achieved at higher doses of alum and lower doses of kaolinite or bentonite. When the settling time increased to three days, the absolute maximum of turbidity removal was achieved at low doses of both alum and kaolinite or bentonite (Fig. 6.16c). The best removal of turbidity with alum and bentonite (85%) was better than that of alum and kaolinite (61%) but was achieved at higher doses of alum when bentonite was used as a coagulant aid.

Fig. 6.15a: Optimum alum and kaolinite doses for 40 minutes settling

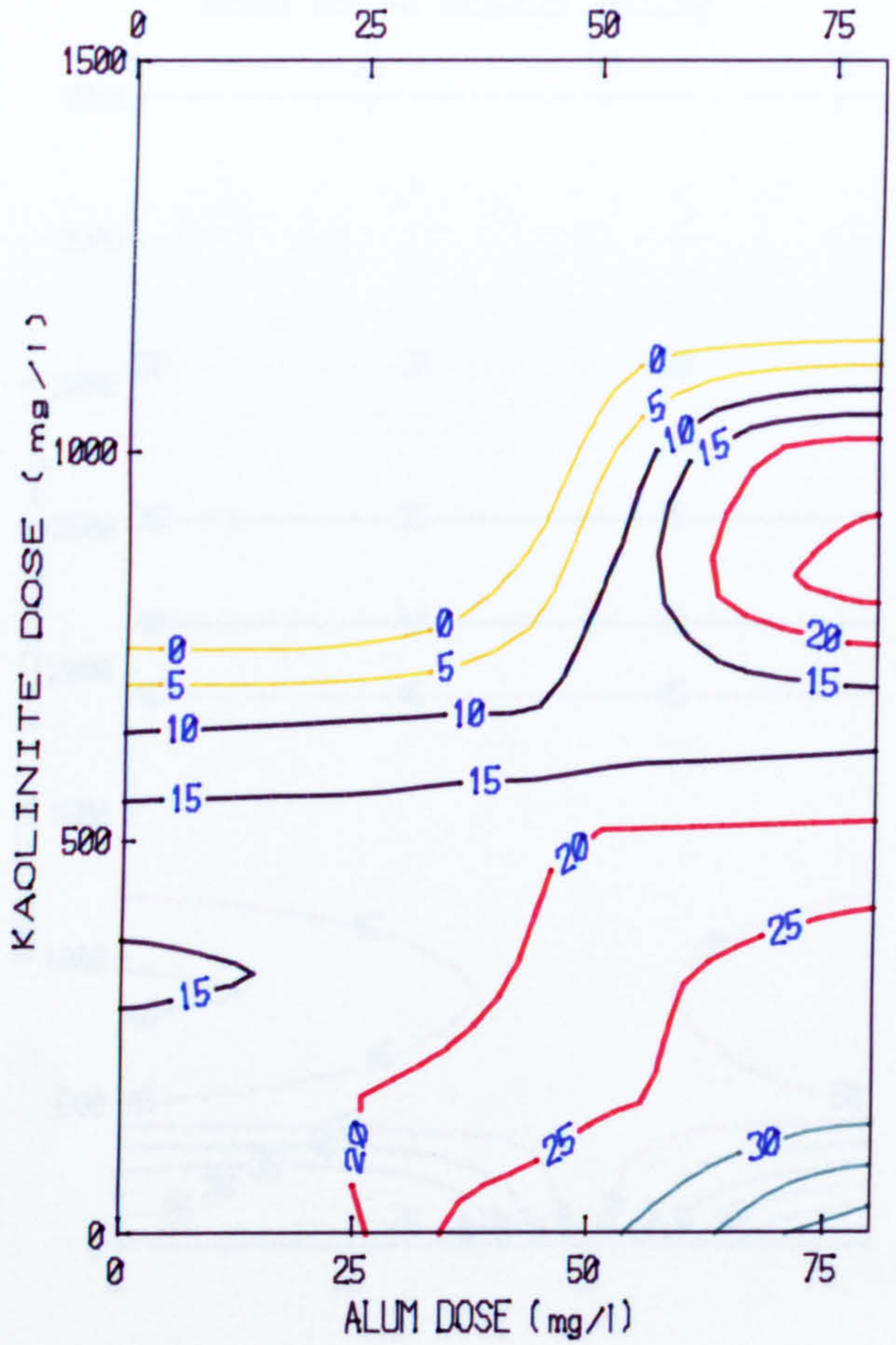


Fig. 6.15b: Optimum alum and kaolinite doses for 1 day settling

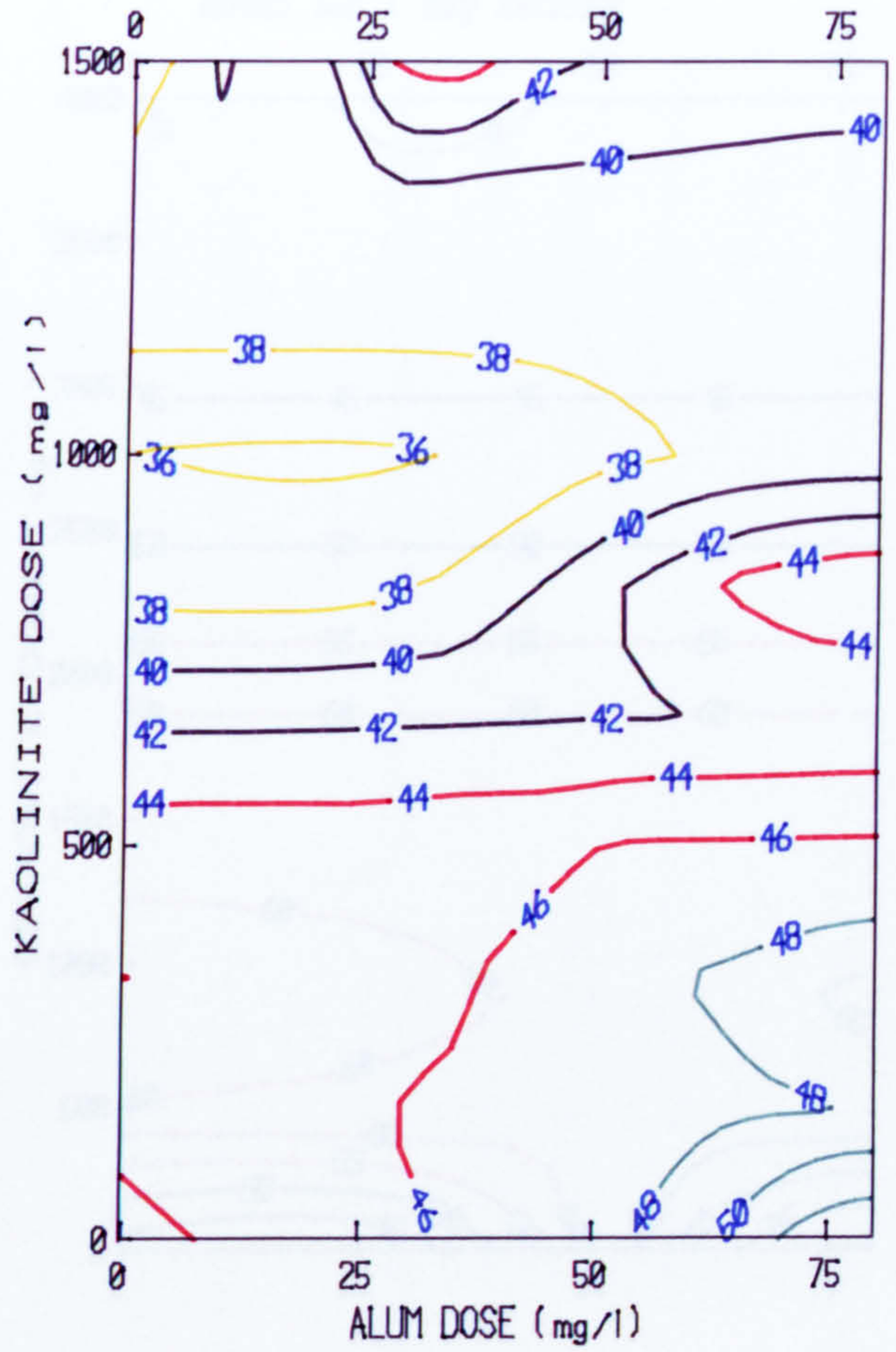
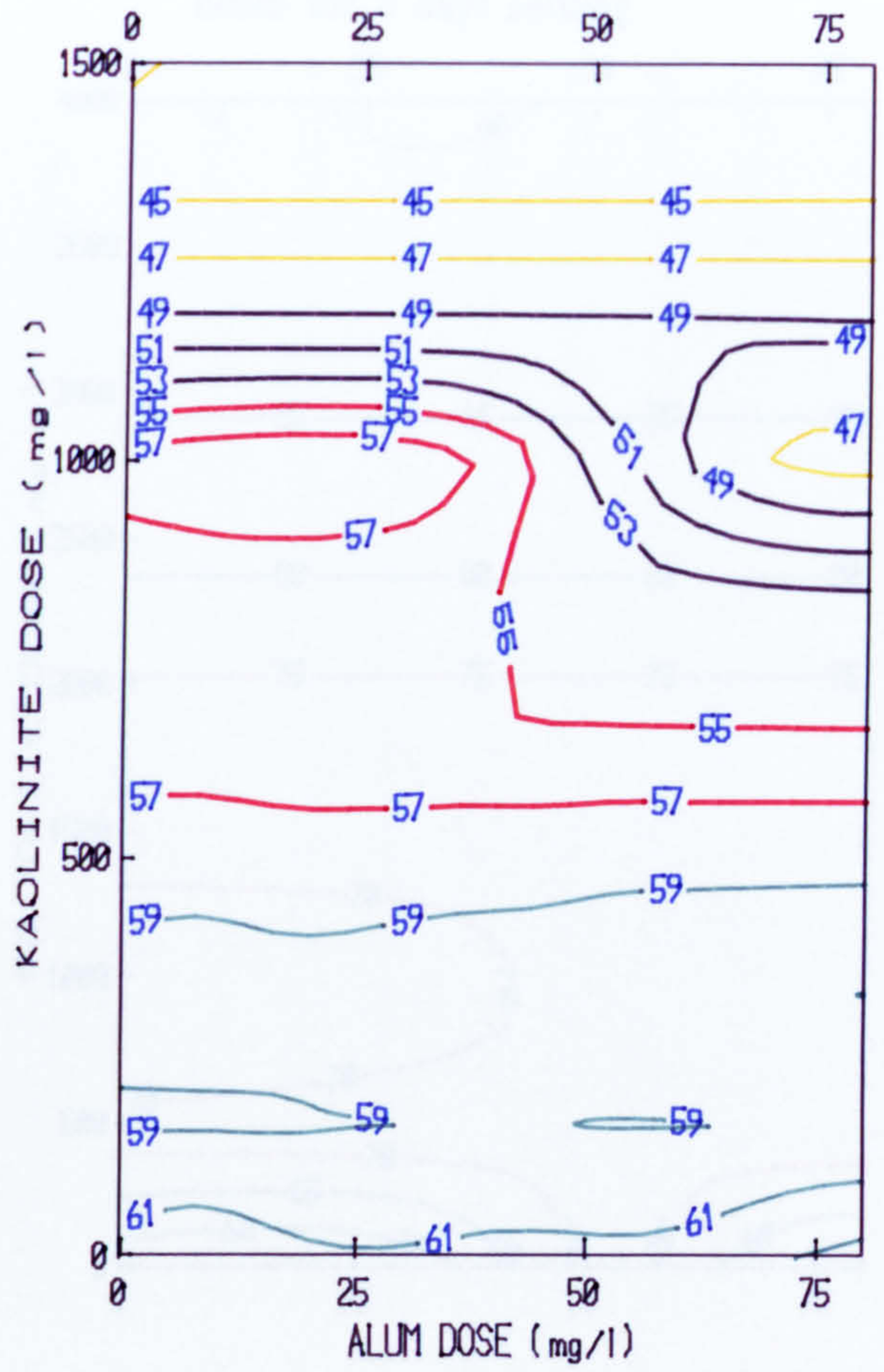


Fig. 6.15c: Optimum alum and kaolinite doses for 3 days settling



**numbers on contours represent percentage turbidity removal

Fig. 6.16a: Optimum alum and bentonite doses for 40 minutes settling

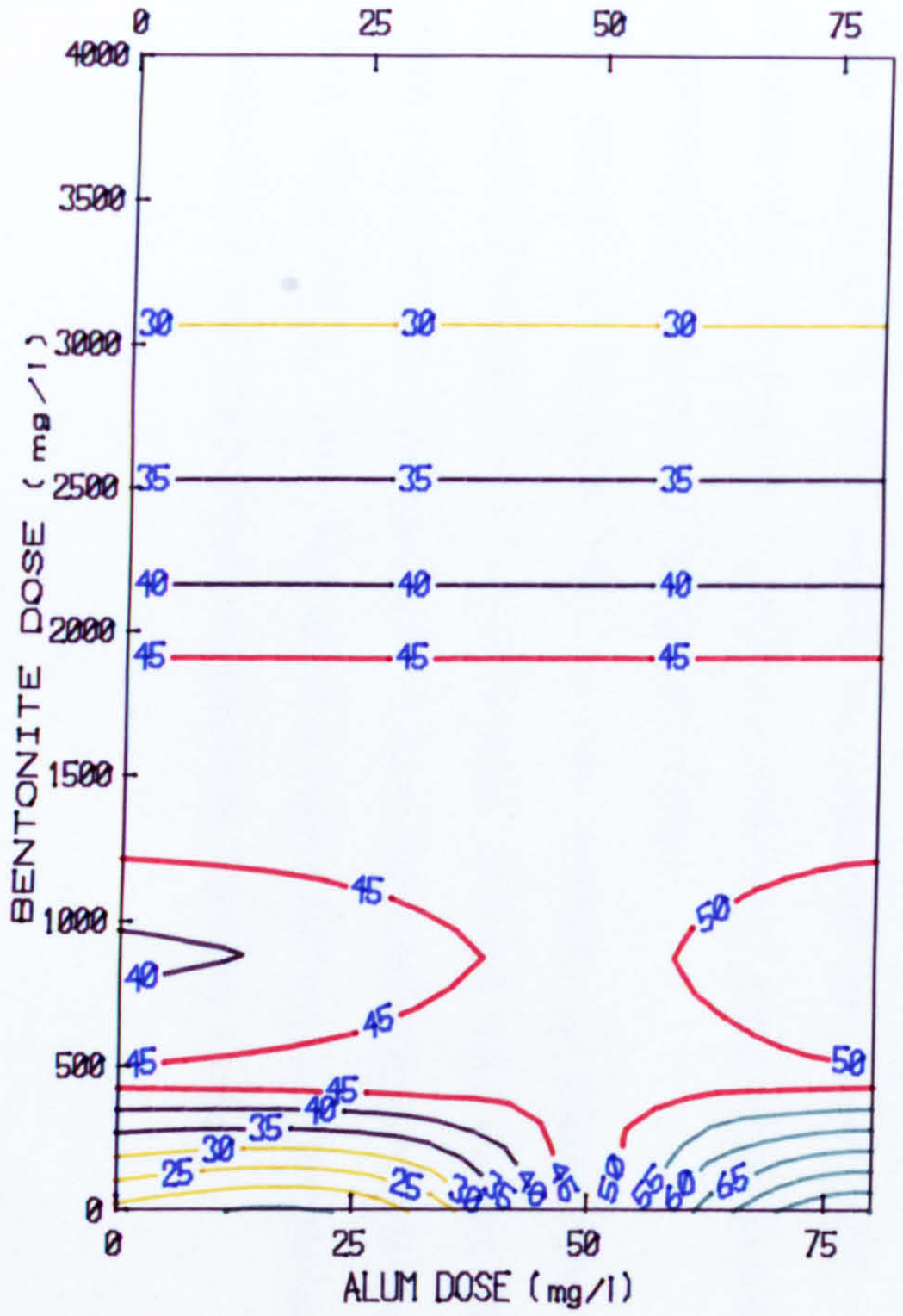


Fig. 6.16b: Optimum alum and bentonite doses for 1 day settling

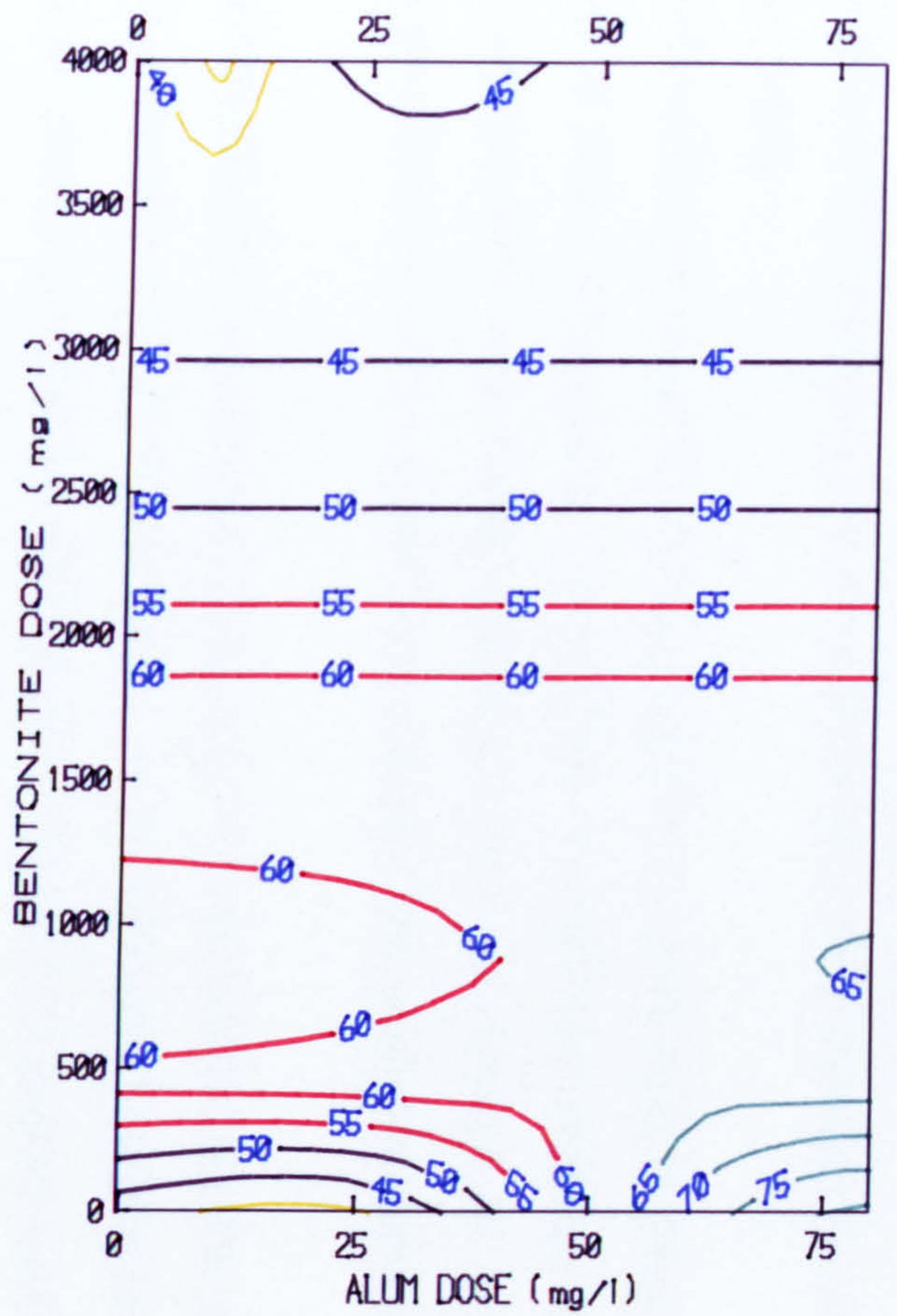
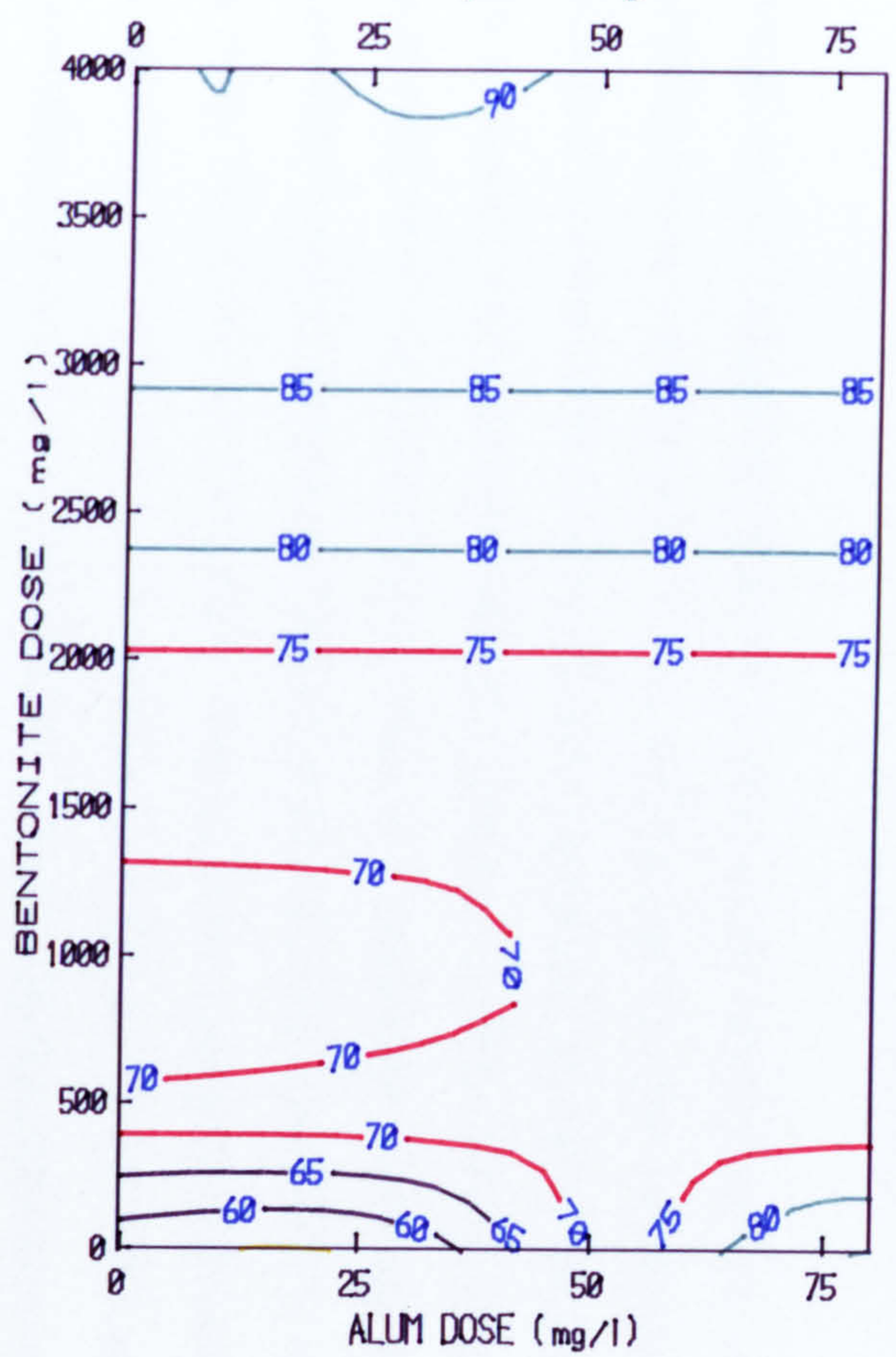


Fig. 6.16c: Optimum alum and bentonite doses for 3 days settling



**numbers on contours represent percentage turbidity removal

6.3.3 Kaolinite and Bentonite as Primary Coagulants for Algae Removal

Although no investigations had been previously carried out on clay for algae coagulation, many studies have been carried out on the effectiveness of clay for removal of bacteria and viruses. Bitton and Mitchell (1974) showed that coliforms are adsorbed onto montmorillonite particles. Olsen (1987) found that bentonite removed 90% of *Schistosoma Mansoni* cercariae even applying a primitive procedure used in Sudanese villages (see Jahn 1976 and 1977). At 1 g/l of bentonite clay, 99-99.9% of *Coxsackie* viruses were removed following the clarification process as performed in Sudanese villages. Also 5-30 mg/l of bentonite were effective in removal of turbidity employing the primitive ways of Sudanese villages (Lund and Nissen, 1986). Similar degrees of reduction of bacteria with bentonite employing the same technique were obtained by Madsen and Schlundt (1989).

Several mechanisms could be acting together or separately in the process of coagulation of algae by bentonite or kaolinite. Possible mechanisms include adsorption, complexation with metals adsorbed onto the surface of the clay, linkage through adsorbed water molecules or cations and enmeshment in sweep floc.

Negatively charged particles of algae may be adsorbed onto the surface of Kaolinite by physical van der Waal's forces or by hydrogen bonding with the H of the organic matter forming the algal surface. Partial protonation could take place around pH 7-8, which may explain the existence of minimum charge density and zeta potential in this region, as reported by Ives (1956, 1959). Adsorption of other microorganisms, such as coliforms, onto montmorillonite particles was reported by Bitton and Mitchell (1974).

The NH_2 group or $R\text{-}CH_2\text{-}COO^-$ on the surface of algae may make complexes with metals forming part of or adsorbed onto the clay surface. Similar mechanisms were reported for adsorption of ammonia and organic matter onto clay (Lance, 1972; Tan, 1982). Also, the same mechanism was reported between the carboxyl group of viruses and metal coagulants (Bratby, 1980).

Metals may be adsorbed onto the surface of clay or algae and link the two materials together. Viruses were reported to be linked to the clay surface through dissolved cations that were adsorbed on the clay surface (Carlson et al., 1968).

In addition to these mechanisms, algal cells may be caught in the settling flocs of bentonite or kaolinite which have formed due to mechanisms of face to face or edge to face interlocking mechanisms (Section 3.7.2-IV). Besides, algae would be partially removed by settling of algal cells themselves.

Dissolution of coagulating metals such as Al^{3+} due to cation exchange reactions with H_3O^+ or other cations (Borchardt, 1977) may help in the coagulation of algae. However, Al^{3+} is found in the free form only under low pH conditions.

Comparing Fig. (5.12a) and (5.12b) with Fig. (5.11) reveals that coagulation by kaolinite doses less than 2000 mg/l resulted in formation of flocs that settle faster than flocs formed by doses of alum up to 80 mg/l. Higher doses of kaolinite (>2000 mg/l) produced flocs that have settling velocities matching those formed by alum doses of 80-160 mg/l.

The distinctive feature of flocs formed by kaolinite coagulation, is heterogeneity of size of these flocs. Some flocs had good settling properties and were removed quickly from suspension. The remaining portion of flocs settled at slower rates. However, settling of these slower flocs contributed to the improvement of the final quality of the supernatant.

Flocs resulting from coagulation with alum were mainly either fast or slow settling ones, depending on the dose of alum (Fig. 5.11).

It should be pointed out, also, that kaolinite dose had a clear effect on the quality of formed flocs. At doses higher than 5675 mg/l, only fast removable flocs formed and increase in settling time did not have a significant effect on the final clarity of the supernatant.

Flocs formed by bentonite (Fig. 5.13a, 5.13b) settled relatively slower than those formed by kaolinite for the same dose of coagulant. Generally, mixed types of flocs were formed with fast and slow settleability at all tested doses.

The dose of kaolinite and bentonite added, affected the final quality of the supernatant. Higher doses resulted in better quality.

Up to 60% removal of turbidity was obtained by kaolinite at 2000 mg/l dose, and 87% removal was achieved at a dosage of 5657 mg/l of bentonite (Fig. 6.17, 6.18).

The removal of turbidity achieved by kaolinite corresponds to those exhibited by 40 mg/l alum. Removal by bentonite was similar to that by 160 mg/l alum. It should be noted that removal of turbidity was achieved by kaolinite and bentonite as primary coagulants only if enough quiescent settling time was given (Fig. 6.17, 6.18). In the cases of kaolinite and bentonite, 3 days of settling time were needed but for alum the removal was achieved after 40 minutes of settling time.

Although Al_2O_3 formed 25.6% of the kaolinite and 13.7% of the bentonite used in this work (Table 4.2), bentonite exhibited a better ability than kaolinite for turbidity removal. This could be attributed to the fact that most of the Al in kaolinite is fixed and not exchangeable because of small spaces between the layers forming the mineral, which prevent cations from leaving or entering between these spaces. On the other hand, spaces between layers of bentonite are large enough to allow ion exchange and more surface area is available for the occurrence of other physical and chemical interactions (Van Olphen, 1963; Tan,1982).

Another merit of bentonite over kaolinite was the presence of 6.3% of CaO in the former, and only 0.1% in the latter. Ca^{++} can be exchanged easily and has a good adsorptive capacity (ibid).

Two tests were carried out to investigate removal of algae by bentonite and kaolinite together. The first was done with 1500 mg/l kaolinite and various doses of bentonite were added to the jars. In the second test, 4000 mg/l of bentonite were added with different kaolinite doses. The percentages of turbidity removal for both tests are shown in Fig. (6.19). By comparison with applying each of these clays separately (Fig. 6.17, 6.18), inferior results were obtained by adding the coagulants together.

Fig.6.17: Removal of turbidity vs kaolinite dose
at different settling times

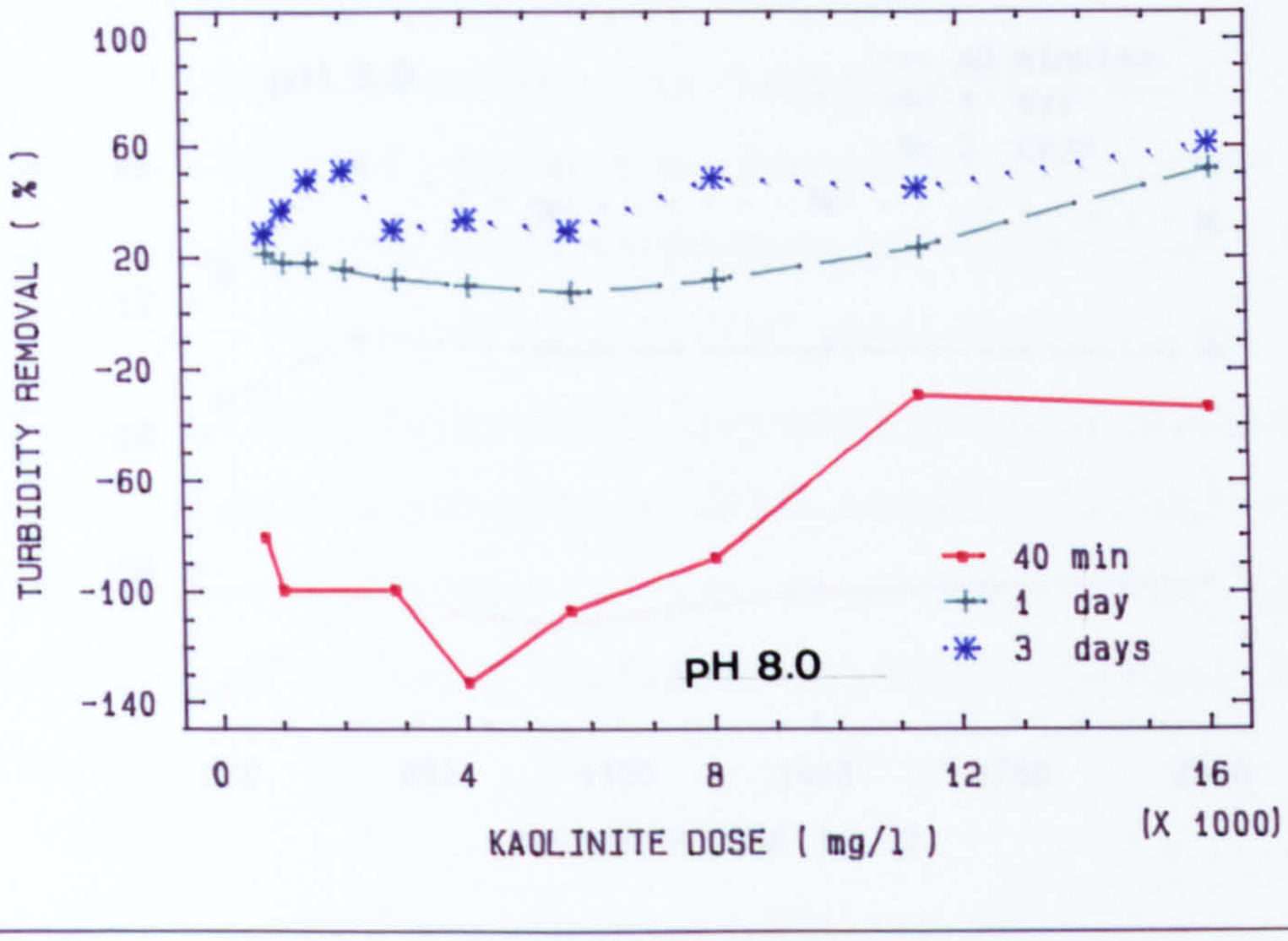


FIg.6.18: Removal of turbidity vs bentonite dose
at different settling times

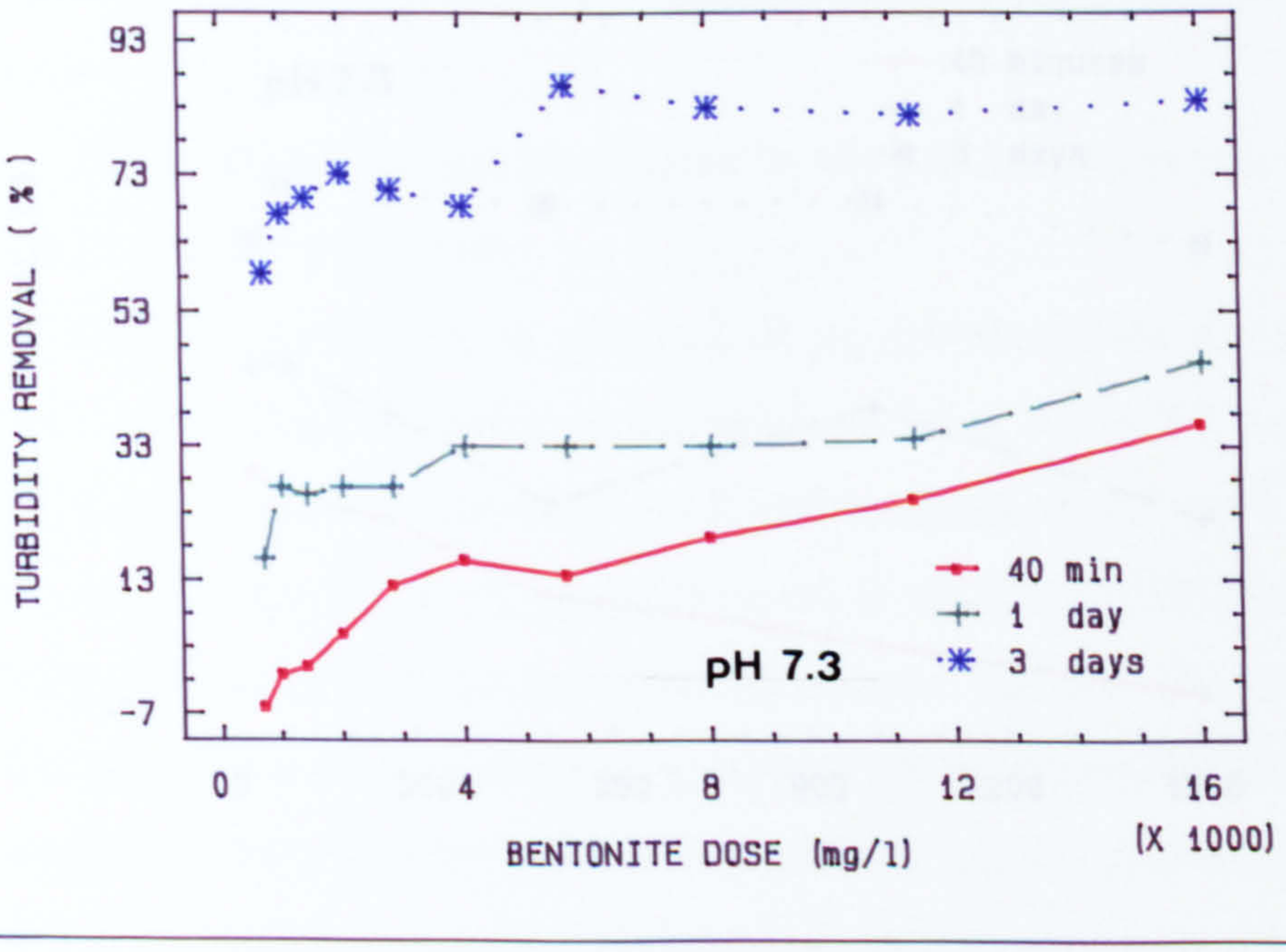


Fig.6.19a: Removal of turbidity at 1500 mg/l kaolinite and various bentonite doses

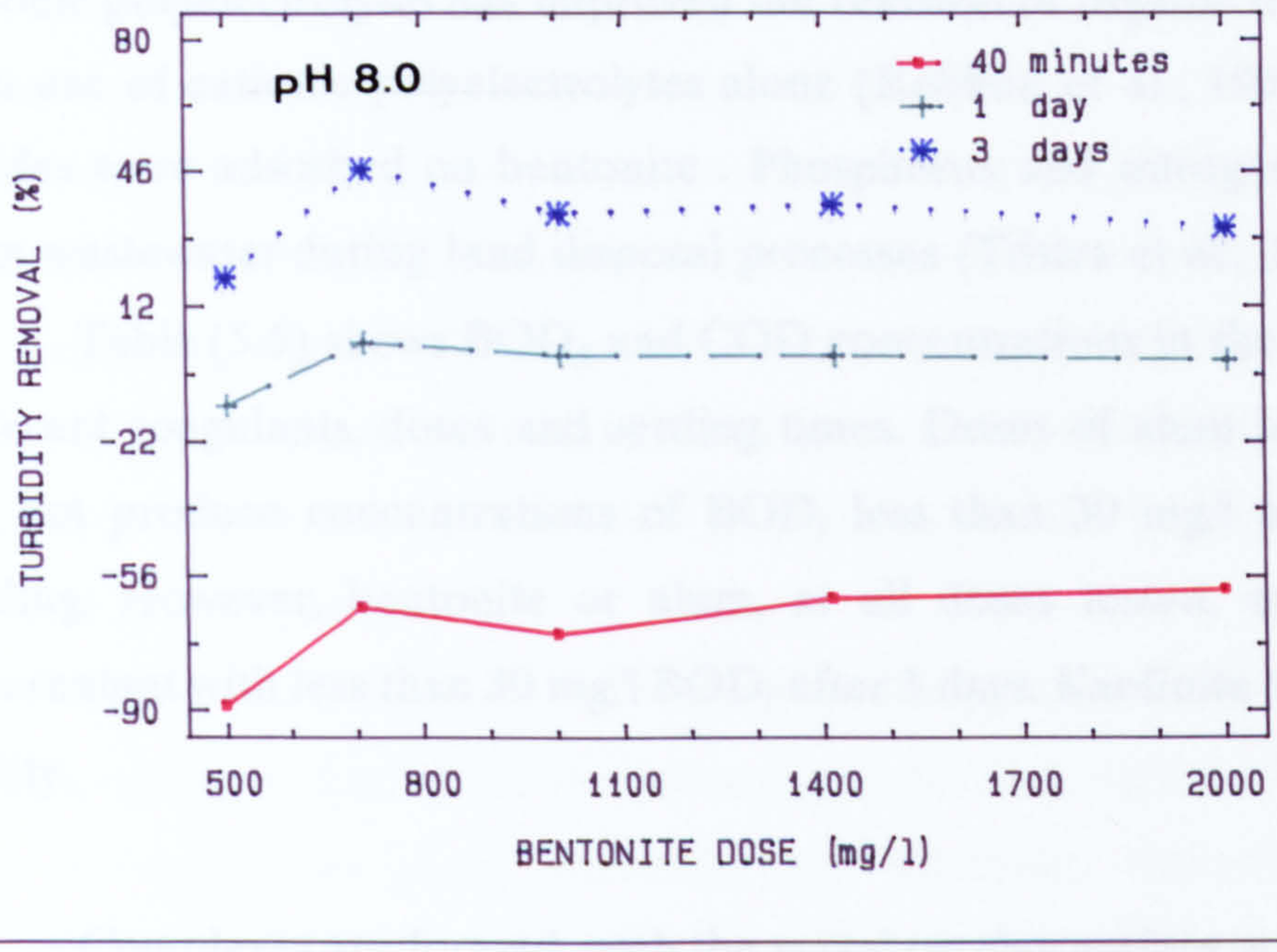
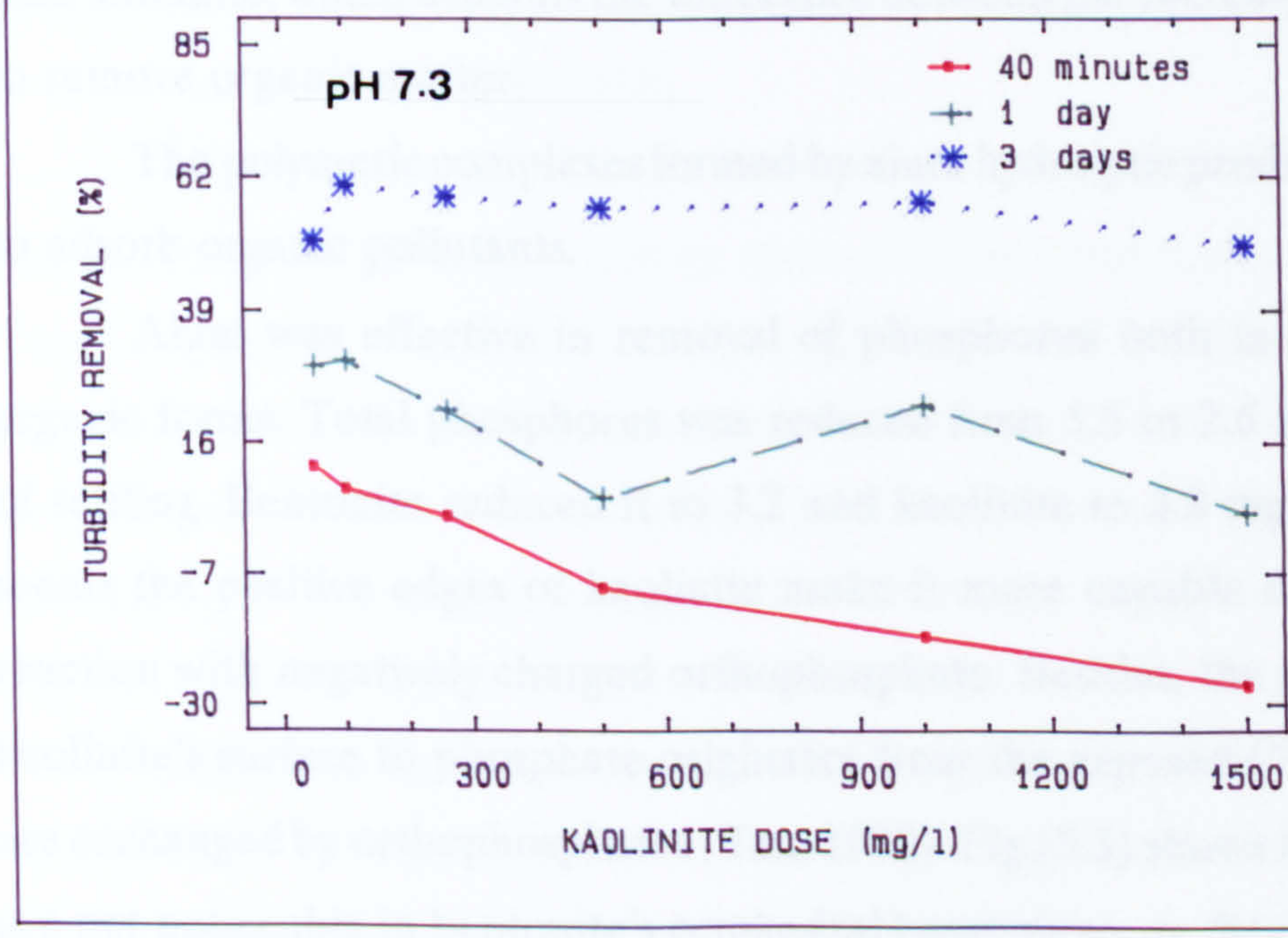


Fig.6.19b: Removal of turbidity at 4000 mg/l bentonite and various kaolinite doses



6.3.4 Nutrient Removal

In addition to removal of turbidity, coagulation was found to remove carbon, phosphorus and nitrogen from treated effluent. Addition of clay to cationic polyelectrolytes has improved the removal of organic matter compared with use of cationic polyelectrolytes alone (Rebhun et al., 1969). Amines and amides were adsorbed on bentonite . Phosphorus and nitrogen were removed from wastewater during land disposal processes (Tilstra et al., 1972).

Table (5.6) shows BOD_5 and COD concentrations in the supernatant for different coagulants, doses and settling times. Doses of alum less than 40 mg/l did not produce concentrations of BOD_t less than 30 mg/l after 40 minutes settling. However, bentonite or alum, at all doses tested, each produced a supernatant with less than 30 mg/l BOD_t after 3 days. Kaolinite did not show this ability.

Complexes are formed, with the metal on clay surface acting as a central ion, and ligands are formed from Cl^-, or $R\text{-}CH_2\text{-}COO^-$ form organic compounds which could be anionic, cationic, or nonionic. Water adsorbed onto the surface of the clay may also react with the functional group of the organic matter.

The large surface area of bentonite gives it higher adsorptive capacity than kaolinite, which explains the difference between the two clays in their ability to remove organic matter.

The polymeric complexes formed by alum hydrolytic products make it able to adsorb organic pollutants.

Alum was effective in removal of phosphorus both in the soluble and organic forms. Total phosphorus was reduced from 5.5 to 2.6 mg/l after 3 days of settling. Bentonite reduced it to 3.2 and kaolinite to 2.8 mg/l (Table 5.7). It seems the positive edges of kaolinite make it more capable than bentonite in reaction with negatively charged orthophosphate. Besides, the strong affinity of kaolinite's surface to phosphate originates from the exposed OH^- groups which are exchanged by orthophosphates (Tan, 1982). Fig.(3.3) shows that these groups are not noticeable in bentonite's octahedral basal plane, as they are in kaolinite.

The degree of removal of phosphates by clay is affected by pH and type and concentration of phosphorus compounds (Jørgensen et al., 1973). In addition to affecting the charge on clay surfaces and edges, pH determines the species of phosphorus present. At pH 1.75, over 50% of phosphoric acid, H_3PO_4, is not dissociated to $H_2PO_4^-$. In the pH range 4-5, $CaH_2PO_4^+$ complex may be formed in the presence of calcium clay (Murray et al., 1985).

At pH 7-8 the species most present are PO_4^{3-}, $H_2PO_4^-$, HPO_4^{2-}, $H_3PO_{10}^{4-}$, $H_2P_2O_7^{3-}$. At pH 9.7 the only species present is the orthophosphate group (Jenkins et al., 1970, 1971). The value of pH during this work (7-8) gives rise to the presence of the negative species of phosphates. The high negative charge density of bentonite compared with kaolinite made the former less effective in removal of phosphates.

The three coagulants showed some ability in removal of ammonia and organic nitrogen (Table 5.7). In decreasing order, their ability of removal of NH_3 may be written as alum, bentonite and kaolinite. Ammonia, NH_3, is in equilibrium with ammonium ion, NH_4^+, in aqueous solution (Eq. 3.5), which makes the positive ion more easily adsorbed onto the larger negatively charged surface area of bentonite, compared with kaolinite.

Ammonia may be removed by complexing with central cations on clay surfaces (Lance, 1972; Tan, 1982). However, this reaction could not be important in the tests of this research because ammonia concentration is negligible at pH < 8 (Sawyer and McCarty, 1978).

Total phosphorus residuals after coagulation are plotted in Fig. (6.20), (6.21) and (6.22) for coagulation by alum, kaolinite, and bentonite, respectively. These Figures show clearly that improvement of total phosphorus removal was generally attained by increase of sedimentation period and increase of coagulant dose. It seems that the rate of settling of formed flocs, and the rate of diffusion of dissolved substances through clay adsorbed water, make removal of phosphorus and organic matter time dependant.

Increase of kaolinite dose above 8000 mg/l did not show any improvement in removal of total phosphorus. It seems that some of the available species of

Fig.6.20: Total phosphorus in the supernatant after coagulation with alum

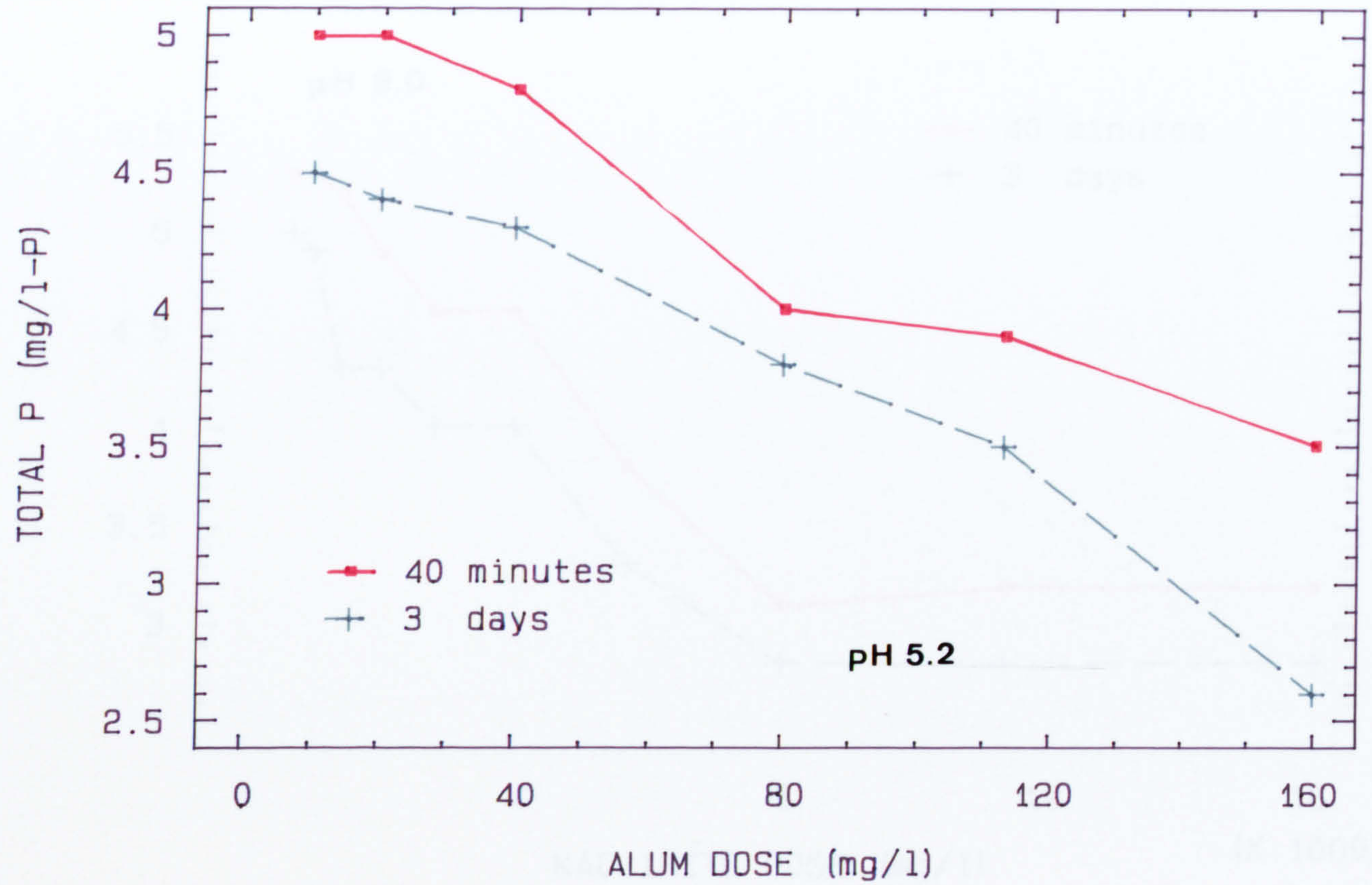


Fig.6.21: Total phosphorus in the supernatant
after coagulation with kaolinite

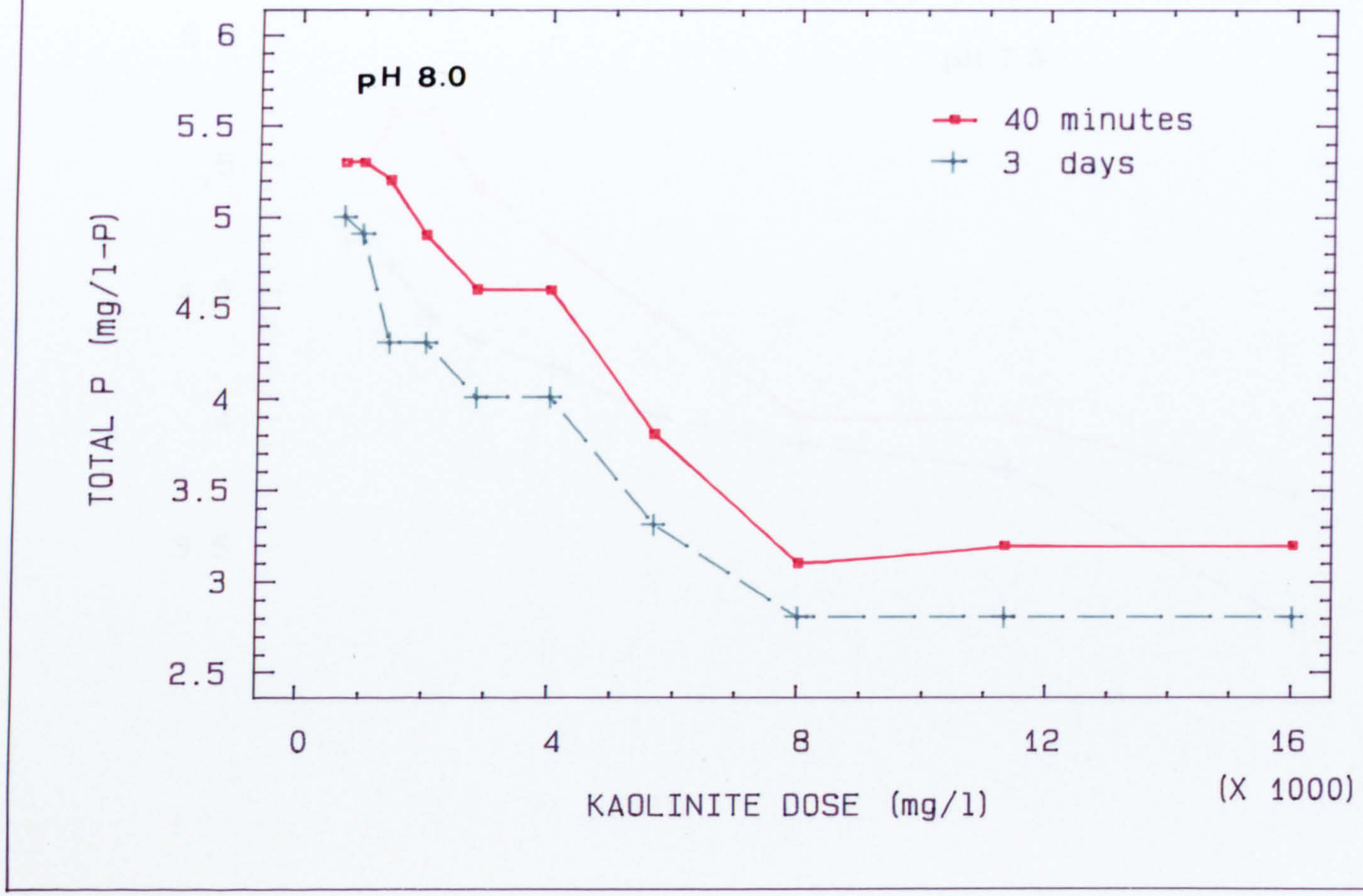
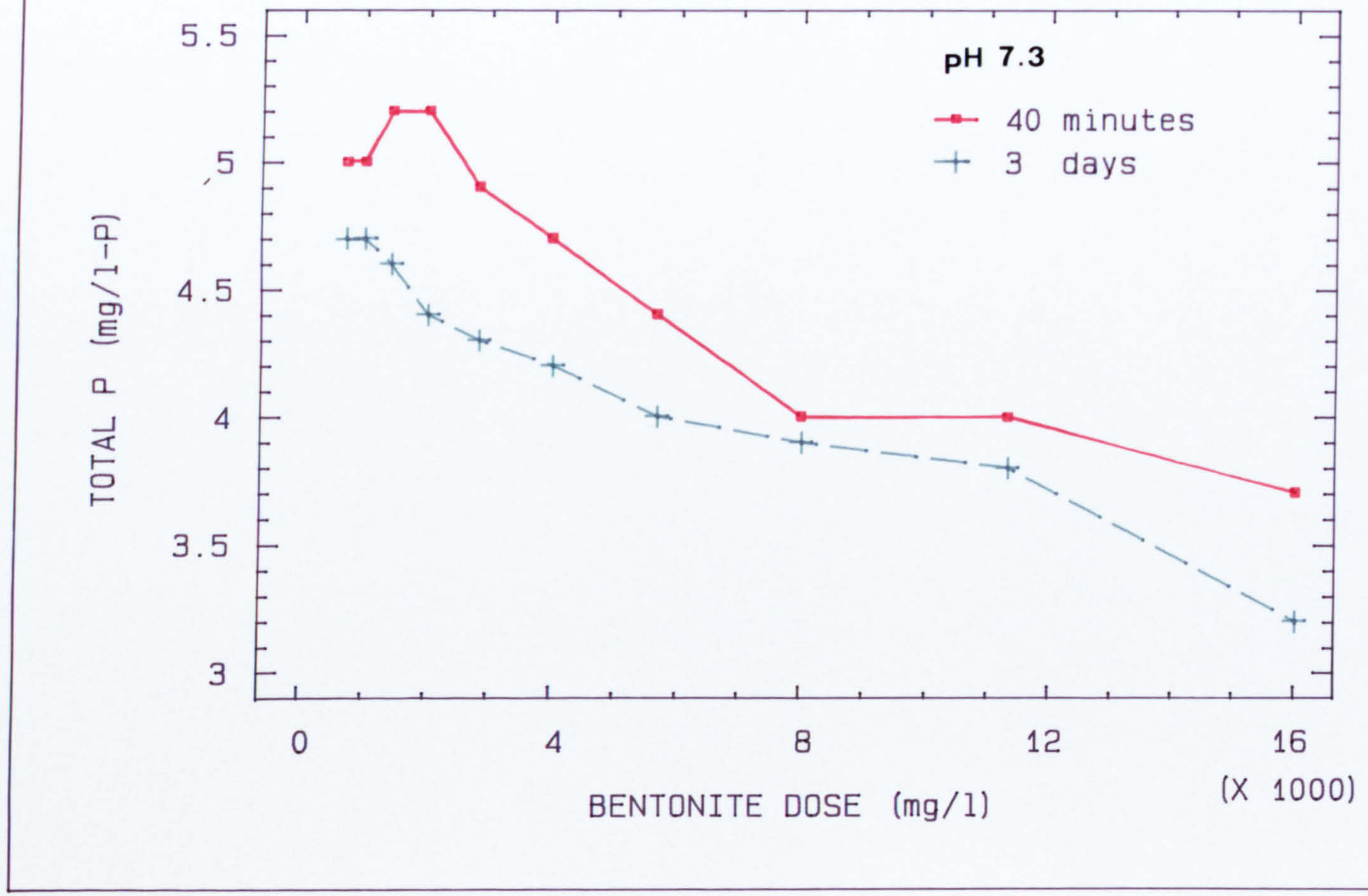


Fig.6.22: Total phosphorus in the supernatant after coagulation with bentonite



phosphorus are not adsorbable on kaolinite. Thus, addition of more kaolinite did not enhance removal of phosphorus any further.

6.3.5 Application of the Jar Test Results

The main aims of the jar test were to determine the optimum coagulant dose, optimum pH and to choose among
coagulants. The optimum dose may not be that which achieves best removal but could be that which gives acceptable end results at lowest cost. Scale-up of the jar test to field scale is the problem that usually faces the engineer in design and operation.

Jar test results may be useful to the engineer in the design of sedimentation tanks when a multiple sampling technique is followed.

(I) Scale-Up of the Jar Test Results

The jar test has been extensively used for monitoring the performance of existing water or wastewater coagulation facilities. Also, it has been used by designers as an aid or substitute for mathematical models, which usually lack the precision to predict optimum dose of coagulant, mainly because of the varying characteristics of waters being treated (Tekippe and Ham, 1970; Stevenson, 1980).

On the other hand, several investigations have pointed out the drawbacks of the jar test with respect to the relationship between field scale unit performance and the results of the test. Small mixers cannot be assumed to be hydraulically similar to large plant scale mixers. Being a batch process, the jar test does not show the short circuiting that occurs in flow-through plant scale (Hudson and Wolfner, 1967; Culp and Culp, 1970; Tekippe and Ham, 1970). This requires that higher coagulant doses should be applied in the field over those predicted by the jar test (Benedeck and Bancsi, 1976). On the contrary, other researchers have found that jar tests overestimate the required coagulant dose (Stevenson, 1980; Lockyear and Stevenson, 1986). They have argued that

higher G values applied in the jar test relative to those in the plants, in order to keep flocs in suspension, result in poorer settlement rates. Jackson and Sheiham (1981) noticed that the jar test overestimates coagulant dose with respect to turbidity removal and suggested a dose corresponding to 95% of that indicated for maximum turbidity removal.

Despite the findings of these workers, that the jar test overestimates or underestimates the dose, the test is still more precise than formulae and it is still used for determining optimum coagulant dose (EPA, 1975; Stevenson, 1980). Some workers have found a good match between jar test results and the field scale. Satisfactory results were found by Jackson and Sheiham (1981) when comparing jar and field scale results of colour removal. Hudson and Wagner (1980) reported that an excellent relationship exists between the test and plant operating records.

The previous discussion shows that the discrepancy between jar test and field scale may be attributed to three sources: degree of turbulence, short circuiting and settling time.

The effect of degree of turbulence delivered during the flash mixing or flocculation step on properties of particles was presented in Chapter 3. In order to rationalize the jar test, Camp and Conklin (1970) recommended that the mean velocity gradient, G, and the time of flash mixing and flocculation in the test should match those in the field for best correlation of results.

If shortcircuiting is overcome or reduced in the field flocculators and sedimentation basins it is expected that the correlation between the test and the field unit performance will increase. Shortcircuiting may be reduced by the introduction of baffles, for example. Hudson and Wolfner (1967) suggested that at least three flocculation basins in series are necessary to control shortcircuiting.

The settling time in the jar test must be relevant to that of the actual treatment plant (Reed and Robinson, 1984). Thus, it is more appropriate for design to collect data at different settling times in the jar test rather than taking one sample at the end of a prespecified period.

(II) Design of Process Units

Three units are mainly required to be designed for coagulation of effluent: a mixing basin, a flocculator and a sedimentation basin.

The turbulence induced in the entrance to the flocculator may be employed for flash mixing. To be consistent with the low cost characteristic of stabilization ponds, flocculators must be chosen also on the basis of cost. Mechanical-power consuming units should be avoided. Hydraulic flocculators, such as baffled channels and gravel bed flocculators, offer a good alternative. Design of hydraulic mixing and flocculating units is principally based on the velocity gradient given in Eq. 3.7. Detailed design may be found in Barnes et al. (1981) and Schulz and Okun (1984).

The data acquired from multiple sampling jar tests serve well for the design of sedimentation tanks. For each plot of fraction of remaining turbidity versus time (e.g. Fig. 5.11), a corresponding velocity distribution curve can be constructed.

Since all the samples were taken at a fixed depth, d, from the surface of the water then the turbidity measured at a certain time, t represents the fraction of solids that have not settled yet after that time, t, or in other words, the fraction that has a velocity less than or equal to d/t. An example of these velocity distribution curves is shown in Fig. (6.23) for coagulation by kaolinite. This Figure corresponds to Fig. (5.12b).

Similar velocity distribution curves were drawn and are presented in Appendix C.

Because the settling depth is short in the jar test, discrete settling can be assumed in the jar (Reed and Robinson, 1984). Thus, the data of the velocity distribution curve may be used for design of settling basins utilizing the overflow rate Eq. 6.2 developed by Camp (1946). Particles with settling velocity greater than or equal to the approach velocity, v (overflow rate), will be completely removed. Particles possessing settling velocity $v_s < v$ will be removed by the ratio v_s/v. If the overall removal is designated by R, then

Fig.6.23a: Velocity distribution at various kaolinite doses

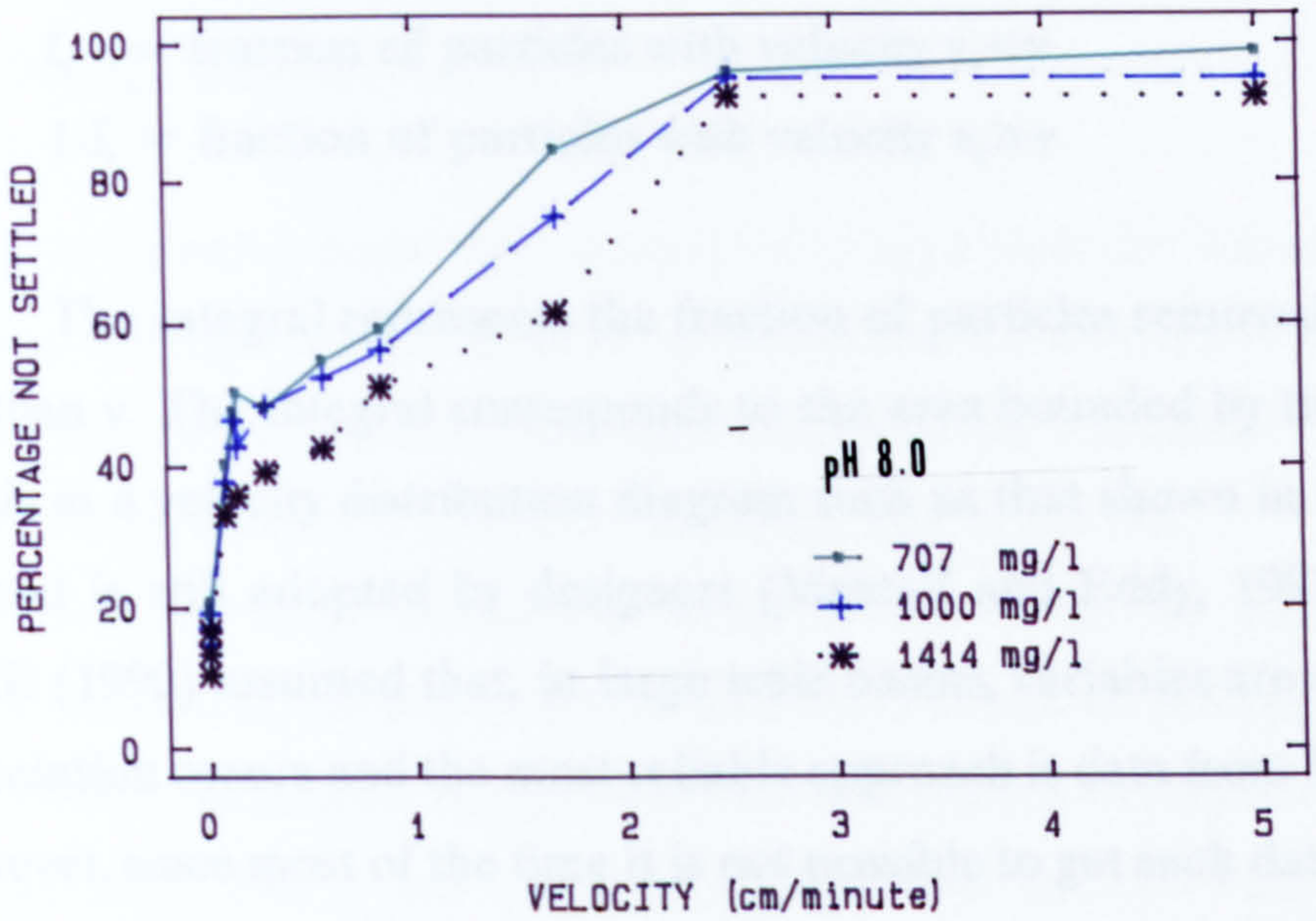
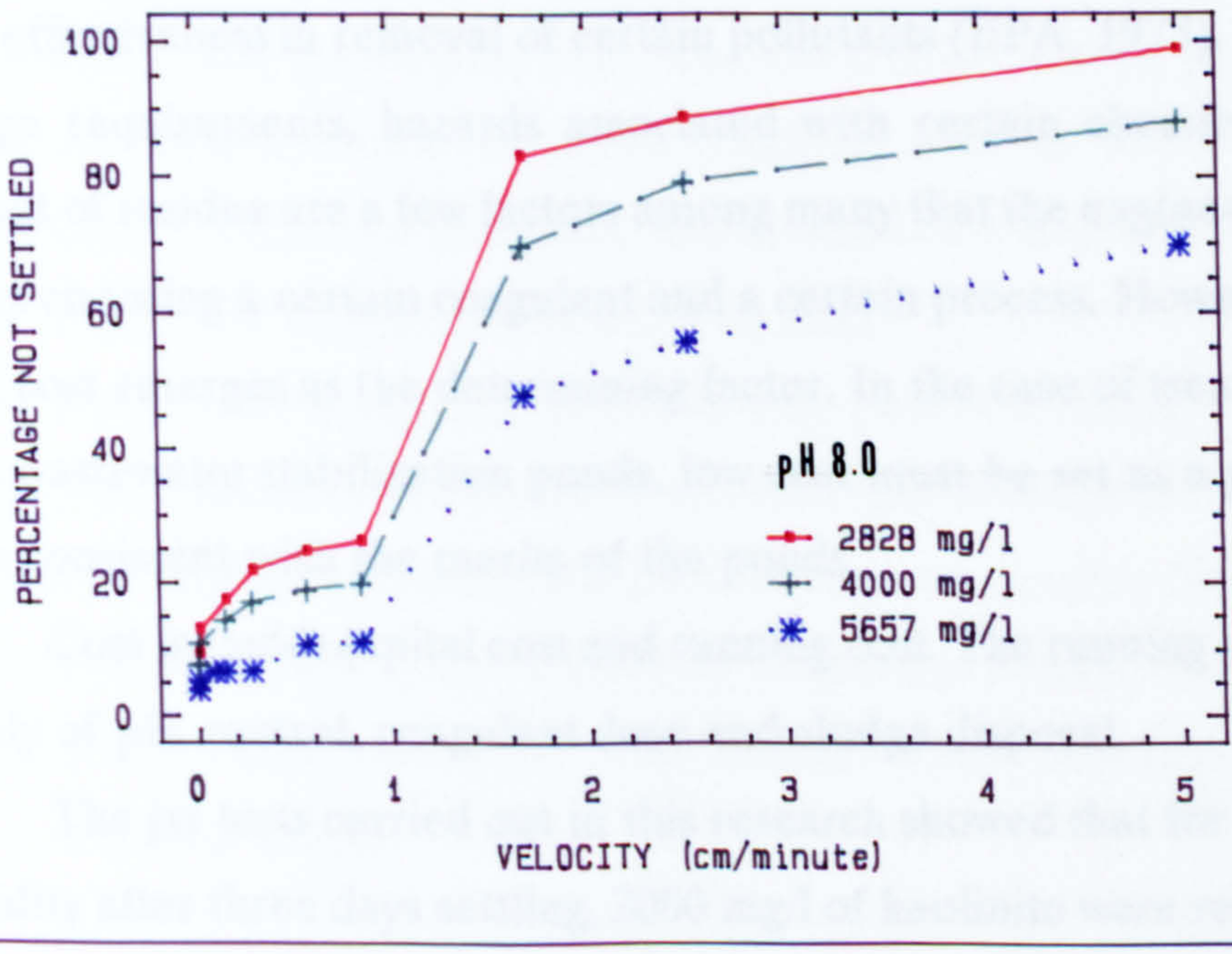


Fig.6.23b: Velocity distribution at various Kaolinite doses



$$R = 1 - f_s + \frac{1}{v}\int v_s df \quad \ldots \quad (6.2)$$

R = fraction removal

f_s = fraction of particles with velocity $v_s < v$

$1-f_s$ = fraction of particles with velocity $v_s > v$

The integral represents the fraction of particles removed with velocity v_s less than v. The integral corresponds to the area bounded by the curve and the Y-axis in a velocity distribution diagram such as that shown in Fig. (6.23). This method is still adopted by designers (Metcalf and Eddy, 1991). AWWA and ASCE (1990) assumed that, in large scale basins, variables are complex, further flocculation occurs and the most reliable approach is data from field scale plants. However, since most of the time it is not possible to get such data they stipulated that laboratory scale tests may be utilized to give a velocity distribution for the water being treated.

(III) Choice of Coagulant and Cost

Jar tests are employed to differentiate between coagulants in terms of their effectiveness in removal of certain pollutants (EPA, 1975). Factors such as storage requirements, hazards associated with certain chemicals, nature and amount of residue are a few factors among many that the engineer must consider before choosing a certain coagulant and a certain process. However, most of the time, cost emerges as the determining factor. In the case of treating the effluent from wastewater stabilization ponds, low cost must be set as a priority in order to be consistent with the merits of the ponds.

Cost includes capital cost and running cost. The running cost is composed mainly of pH control, coagulant dose and sludge disposal.

The jar tests carried out in this research showed that for 60% removal of turbidity after three days settling, 2000 mg/l of kaolinite were required. The cost of coagulant will be 4.4 Jordanian Dinar (£ 1=JD 1.22) per 1000 m^3 of effluent

treated. For the same degree of treatment, 60 mg/l alum and three days of settling were required. The cost of alum will be JD 13 per 1000 m^3 of effluent treated. These calculations were based on 1991 prices in the Jordanian market at JD 215/1000 kg alum and JD 2.2/1000 kg kaolinite.

6.3.6 Sludge Disposal

In general, increased amount of sludge and the lessening of disposal options, brought about in part by an increased awareness of the polluting potential of these materials, has made this phase for wastewater management the most critical aspect for many communities and industries.

For the three coagulants tested in this work, the maximum depth of sludge noticed in the jars was 1 cm after 40 minutes of settling. Although more solids settled with time, sludge depth was noticed to be compressed with a longer settling time. For 1 cm depth of sludge, and 11.5cm diameter of jar, the maximum amount of sludge produced would be 52 cm^3/l of effluent treated (0.052 m^3/m^3).

Sludge disposal alternatives may be categorized into utilization and non-utilization disposal. Utilization refers to the beneficial use of sludge or sludge by-products. Sludge may be used as soil amendment: application to agricultural land for its fertilizer value; reclamation of disturbed lands such as construction sites, strip-mined lands, gravel pits, and clean-cut forests. Sludge may be used as a source of heat; a source of useful products such as wastewater treatment chemicals, industrial raw materials, and construction materials; as animal food; and as a compost with solid waste (EPA, 1983).

Wastewater sludge may not always be used as a resource because of land acquisition constraints or because it contains high levels of metals or other toxic substances. Sludge disposal options that do not involve beneficial use are referred to as disposal or non- utilization-disposal processes. These include ocean disposal, landfilling, dedicated land disposal (land farming) and deep well injection (EPA, 1979).

Algal sludge may be utilized for agricultural land application for its fertilizer value and conditioning of the soil. This option is encouraged when bentonite and kaolinite is used as coagulant. However, if alum is used this could be a constraint because alum in drinking water has been associated with certain neurological disorders, such as dialysis dementia and Alzheimer's disease. Ingested aluminium accumulates in the bones, whereas aluminium compounds inhaled accumulate in the lungs and lymph nodes (WHO, 1984). However, aluminium has not been mentioned in the guidelines for sewage sludge application to agricultural land. Hucker (1980) summarized the guidelines from the U.S, Canada, United Kingdom, the Netherlands, Germany, France, Switzerland, Denmark, Finland, Norway, and Sweden.

Elkins et al. (1985) and Gersberg et al. (1988) found that sludge produced by treatment with clay as adsorbent followed by alum flocculation is easy to dewater and was used in the production of lightweight concrete and ceramic materials by firing at 1093°C. On the other hand, alum sludges were found difficult to dewater (Hannah, 1971).

Although not a final disposal process, recycling of sludge to a primary stabilization pond that receives raw sewage is an alternative for temporary sludge management in pond systems. Such a process did not adversely affect the performance of the primary pond when sludge was recycled for three months in the work of Parker (1966). The lagoon receiving recycled sludge was compared with a control lagoon and no significant difference was noticed between the performances of the two ponds.

6.4 Crossflow Microfiltration

Crossflow filtration is differentiated from dead end or cake filtration by the direction of flow with respect to the membrane. In the former, the liquid to be treated flows parallel to the surface of the filtration membrane where, in the latter, the flow direction is perpendicular to the membrane surface (Murkes and Carlsson, 1988).

Flow parallel to the membrane surface has been used for a long time in ultrafiltration and reverse osmosis. However, in the engineering literature, crossflow filtration usually refers to microfiltration. Reverse osmosis captures macromolecules and salts (less than 0.001 μm) an allows water to pass through the membrane; ultrafiltration retains macromolecules (0.001-0.1 μm) and allows water and dissolved salts to pass through the membrane; microfiltration removes cells, colloids, suspended solids and some selected large size macromolecules (0.1-10 μm) while water and salts pass through the membrane (Cartwright, 1986; Murkes and Carlsson, 1988).

Crossflow microfiltration allows the production of clear filtrate (permeate) by separation of very fine particles at lower cost than dead end filtration. In the latter, clogging by fine particles is a major problem unless coagulants are used . Thus, higher filtration rates are achieved using crossflow microfiltration compared with cake filtration (Henry and Allred, 1977; Murkes and Carlsson, 1988).

The cross section of the crossflow filtration membrane, is manufactured in different modules: hollow fibre, tubes, pleated, flat sheet, and spiral wound sections (Fell, 1986; Belter et al., 1988). The advantages and disadvantages of the different membrane modules have been summarized by Harper (1981), Belter et al. (1988) and MacNeil and McCoy (1989).

Different materials are used to manufacture microporous membranes, including ceramics, sintered calcined porous carbon covered with zirconium oxide and polymers (Murkes and Carlsson, 1988).

The material and geometry of the membrane determines the operational parameters: flux, clogging and ease of cleaning. It was shown experimentally that clogging of the membrane is more probable when pore size is large. Although a larger pore size gives high flow rates at the beginning, after a certain time of filtration solids will penetrate the pores and cause clogging unless the pores are very large such that the quality of filtrate is affected. It was found that a pore size of 0.1-2 μm gave an optimum flux rate (Murkes and Carlsson, 1988).

6.4.1 Theoretical Background of Crossflow Microfiltration

Models which have been set to describe the flux in crossflow microfiltration, have been related to the type of dynamic membrane that is formed due to retention of solids or large molecules by the membrane.

The first type of dynamic membrane is the concentration polarization membrane which is formed due to the accumulation of solute macromolecules at the membrane surface (Blatt et al., 1970, Tanny, 1979). The concentration of this layer is designated C_w and continues to increase until an equilibrium gel concentration is reached, C_g. The flux through the membrane has been expressed as follows (Michaels, 1968; Defilippi and Goldsmith, 1970; Tanny, 1979):

$$J = K_m . \ln \frac{C_w}{C_b}$$

Where, K_m is the back diffusion mass transfer coefficient and C_b is the bulk solution concentration.

Also, the flux, J, has been related to the pressure drop between upstream and the filtrate, ΔP, and membrane resistance, Rm, and the hydraulic resistance of the polarization layer, Rp, (Michaels, 1968; Fane et al., 1981; Cheremisinoff and Azbel, 1983).

$$J = \frac{\Delta P}{(R_m + R_p)}$$

The second type of dynamic membrane is formed by colloidal particles bridging over the pore size, which may be larger than the size of the colloid by an order of magnitude up to 1-2 μm. Some small particles will be deposited inside the pores. In this case, filtration is governed by Ruth's law for cake filtration (Cheremisinoff and Azbel, 1983):

$$\frac{t}{V} = \frac{\mu \alpha C_b}{2 \Delta P A^2} \cdot V + \frac{\mu R_m}{A \Delta P}$$

Where,

t = time of filtration, s

V = volume of filtrate in time t, m^3

μ = dynamic viscosity, $N.S/m^2$

α = the specific cake resistance; resistance of a unit mass deposited in a unit area

C_b = concentration of particles in the bulk solution, kg/m^3

ΔP = pressure drop through the membrane, Pa

A = surface area of membrane, m^2

R_m = membrane resistance, $Pa.hr.m^2/m^3$

From previous models it is clear that flux increases with pressure drop. However after a certain value of ΔP, any increase in pressure drop will not affect the flux. Flux is affected by ΔP when $C_w < C_g$. However, when $C_w = C_g$, any further increase in pressure will compact the dynamic membrane and decrease its permeability (Murkes and Carlsson, 1988).

Velocity affects the shear stress imparted over the dynamic membrane. At low flow velocities ($v \rightarrow 0$) pressure flux is proportional to $v^{3/2}$. For higher flow rates, the thickness of the dynamic membrane becomes very thin and J will be a function of membrane resistance only. At high v values the rate of increae of flux with increase of v is negligible, and high values of flow velocity may prove uneconomic.

Although from the previous equations, the flux, J, should reach a constant value when $C_w = C_g$, it has been found that flux keeps declining with time (Blatt et al., 1970). This has been attributed to increase of hydraulic resistance of the deposited membrane by continuous enrichment with finer particles even though the thickness of the dynamic membrane remains constant (Baker et al., 1985).

Theoretically, the flux becomes dependent on the mass transfer coefficient, k_m, and the feed concentration, C_b, when $C_w = C_g$. However, Murke

s and Carlsson (1988) found that crossflow microfiltration is not significantly sensitive to concentration of solids in the feed. Filtrate purity was almost independent of concentration of solids in the feed and decline of flux occurred in small proportions at moderate concentrations.

6.4.2 Discussion of Crossflow Microfiltration Results

The permeate flux (Table 5.8) is plotted against time in Fig. (6.24a) for runs 1, 2 and 4. This Figure shows that run 1 gave the best flux (75 l/m^2.hr) after four hours of operation, whereas flux dropped to about 40 l/m^2.hr in runs 2 and 4. The three runs were carried out under the same conditions except that in run 1 algae mats formed on the surface of the facultative pond, while no such mats were noticed when runs 2 and 4 were carried out. It seems that new small algal cells, being in a dispersed form, penetrated the fabric of the membrane and resulted in a lower flux in runs 2 and 4. It could be argued that such penetration of cloth pores by small algal cells did not take place when algal mats formed, which resulted in a higher flux in run 1.

In Fig. (6.24b), the change of flux with time in run 2 is overlaid on Fig. (6.24a). The former Figure reveals that run 3 had a flux vs time diagram identical to that of run 1. The high flux in run 3 may be attributed to the coagulation of algae, effected by addition of an alum dose of 10 mg/l to the feed, which prevented the pores of the membrane being clogged by dispersed algal cells.

Averaging the values of turbidity during the crossflow microfiltration experiments (Table 5.8), gives a percentage reduction of turbidity of 71%, 35%, 48% and 49% for the four runs, respectively. It is noticed that best removal of turbidity accompanied the existence of algal mats in the feed pond. In run 3, when an alum dose was added, the degree of turbidity removal was not very much different from runs 2 and 4. An explanation of this may be the possibility of escape of some very small algal cells with the permeate because the small 10 mg/l alum dose might not have been large enough to flocculate the algal cells. Or, it could be argued that in run 4 a partial autoflocculation had occurred, which may be concluded from the difference of flux vs time diagrams between

Fig.6.24a: Flux vs time for runs 1, 2 and 4
in the crossflow microfiltration tests

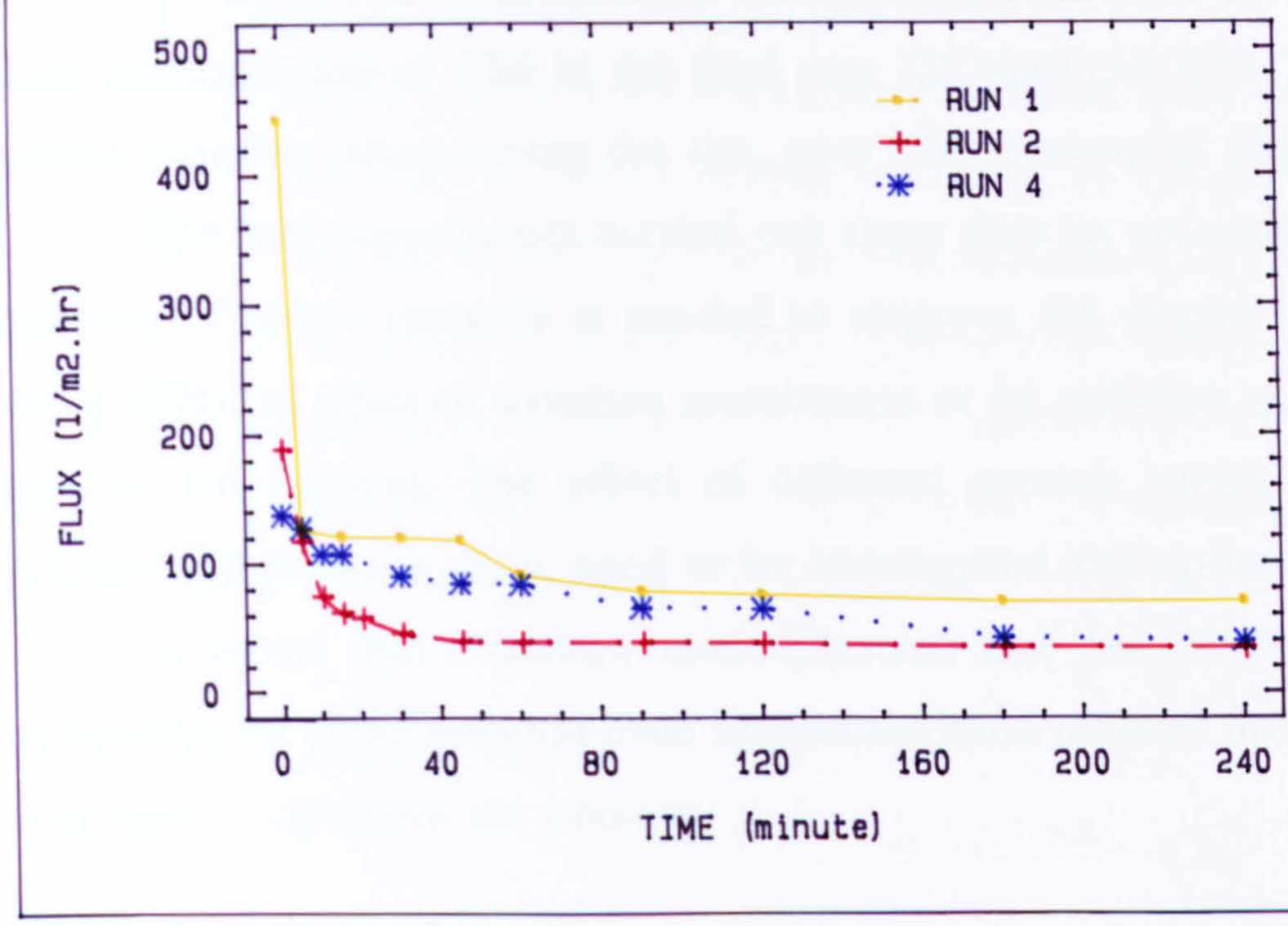
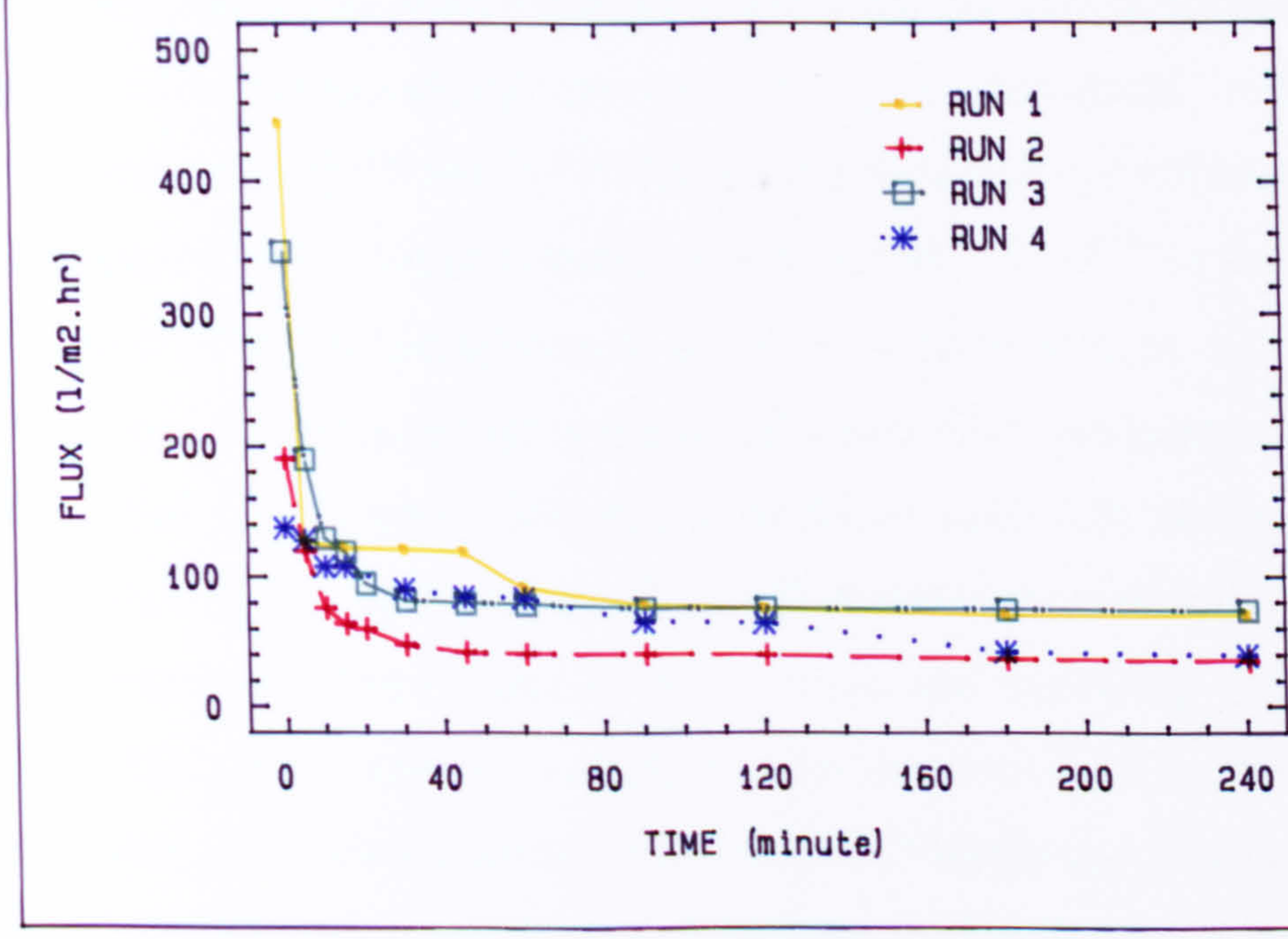


Fig.6.24b: Flux vs time for four runs in
the crossflow microfiltration tests



runs 1, 2 and 4. This partial autoflocculation resulted in a 49% turbidity reduction, which is almost equal to the 48% degree of turbidity reduction attained by the partial flocculation with alum in run 3, but is higher than the 35% turbidityremoval obtained in run 2 in which algal cells were dispersed.

In run 3, TSS concentration was measured for both feed and permeate. The concentration of TSS in the feed was 125 mg/l (25 NTU). An average of eleven samples, taken during the run, gave TSS removal of 66% .

The few experiments carried out show that an acceptable flux rate was achieved. Further research is needed to improve the degree of algae removal using different types of dynamic membranes or by addition of different coagul ants to the influent. The effect of different process variables, such as flow velocity and pressure drop, need to be investigated and optimized.

It seems that crossflow microfiltration may be considered as a viable alternative for algae removal from facultative pond effluent but further research is needed to optimize the process.

CHAPTER 7

Modelling an Upflow Rock Filter for Algae Removal from Facultative Pond Effluent

The procedure for design of rock filters has been dependent on the choice of a suitable volumetric hydraulic loading ($m^3/m^3.d$). The results of this research have proven that this would lead to an overestimation of the filter removal capacity. Actually, at any particular time, algae removal occurs only within a part of the filter height rather than over the whole volume, as the volumetric loading assumes. Thus, design criteria which take into consideration the effect of height on filter performance is required.

7.1 Existing Filtration Models

It was demonstrated in the previous Chapter how rock filter performance and the prevailing mechanism of removal of algae are affected by the physical properties of the algal species present, such as diameter and specific gravity, together with operating variables of the filter, such as approach velocity and filter media size. It was shown that algae removal is proportional to $d_p^{0.098}$ when interception is the mechanism of algae removal and is proportional to $d_p^{0.187}$ when algae removal by gravity settling is dominant. Algae removal is proportional to $d_c^{0.098}$ and $d_c^{-0.134}$ for interception and hydrodynamic mechanisms, respectively. Also, algae removal is proportional to $v^{-0.187}$ in the case of settling and to $v^{-0.134}$ when hydrodynamic action is more important. Similar variations of the effects of different operating variables on filter performance were found by Yao et al. (1971) when different suspensions and filter media were used. This explains why the results obtained by different researchers in the field of filtration have disagreed when different suspensions and operating variables have been used. Thus, it is difficult to adopt a generalized model for rock filters using the microscopic approach mainly developed by O'Melia and Stumm (1967) and Yao et al. (1971), which was reviewed in Section (2.4.4) of this work. There is no

unique relationship between efficiency of removal and the variables used for design, such as d_c and v.

There are two major difficulties in application of Ives' model, discussed in Section (2.4.2) of this work. The main difficulty is the use of specific deposit and concentration in volume/volume units. These are difficult to measure and are not easily related to the most commonly used concentration of mg/l. Furthermore, four constants, f, i, j and Ω_u, must be determined experimentally for each filter bed and flow rate whenever a filter is to be designed (O'Melia and Stumm, 1967; Cleasby, 1972).

The use of Eliassen's model requires the experimental determination of the ultimate mass deposit q_u, which may be very difficult in the case of rock filters and requires that operation be extended for a very long period of time before q_u is reached. However, the basic principle of the model, that removal probability decreases with passage through the filter depth, is realistic. Also, Eliassen's equation does not take into consideration the improvement in performance due to increase of surface area available for deposition, by accumulation of deposits, which was shown experimentally by Ives and Sholji (1965), Ives (1975a) and Amirtharajah and Wetstein (1980). It only considers the deterioration of efficiency caused by an increase of interstitial velocity as a result of reduction in pore size due to deposited material.

The model of Williamson and Swanson for design of rock filters assumed that settling was the dominant mechanism in filter pores and other mechanisms were disregarded. In this research it has been shown that this assumption is not valid. Other mechanisms may become operative and it was found that hydrodynamic action is nearly as important as gravitational settling. Besides, the application of this model needs to define a settling curve for algae and the settling distance d_s, which depends on the pore size of the rock filter. Swanson and Williamson (1980) recognized the difficulty of finding d_s and stated that the rock filter is composed of a series of small settling chambers with varying settling depths. They could not correlate d_s with any property of the rock filter that could be determined and measured physically. Also, design based on overflow rate, which implies removal of particles with settling velocity, v_s, less than approach

velocity, is invalid for upflow filters. These particles will be kept in suspension and escape with effluent unless a change in their characteristics, such as flocculation, takes place (Brown, 1971) or other mechanisms are involved in their removal. In this research it was shown that other important mechanisms were involved, and flocculation, auto and orthokinetic, is thought to take place based on the fact that most TSS were removed in the first half metre of the rock filters used.

7.2 Assumptions and Development of the Model

To describe the variation of solids with respect to depth, Iwasaki's equation (1937) is employed (Eq. 2.6). It states that for any point in the filter the removal per unit depth of filter is proportional to the local concentration of suspended solids. Assuming that the concentration is only a function of distance, and at any point within the filter the concentration does not change with time, Iwasaki's equation can be written in the ordinary form:

$$\frac{dc}{dx} = -\lambda C \quad \ldots\ldots\ldots (7.1)$$

Eq. 7.1 with constant λ through the filter bed did not fit the data from set one, which was taken during a period when one species of algae prevailed. Furthermore, looking at Fig. (6.6) and Fig. (6.7) reveals that most of TSS removal occurred within the first half metre of the filter and the rate of decrease in suspended solids concentration within the rest of the filter height was very small. Therefore, it is reasonable to assume that λ, which represents the probability of removal of particles, decreases with passage through the filter or with increase in degree of removal. This assumption was expressed mathematically by Tchobanoglous and Eliassen (1970) in Eq. 2.20, repeated here as Eq. 7.2:

$$\frac{dC}{dx} = -\left[\frac{1}{(1+ax)^n}\right]\lambda_o C \quad \ldots\ldots\ldots\ldots (7.2)$$

Fig. (7.1) shows the change of retardation coefficient (the term between brackets in Eq. 7.2) for different values of **n**. When the value of **n** is zero, Eq. 7.2 reduces to Eq. 7.1 with $\lambda=\lambda_o$=constant. This occurs with a suspension of particles of one size and filter of unisize bed material.

Solving Eq. 7.2 for concentration of TSS equals **C** at height **x**, and C_1 at height x_1, yields:

$$\frac{C}{C_1} = EXP\left[\frac{-\lambda_o}{a(1-n)}\left\{(1+ax)^{1-n} - (1+ax_1)^{1-n}\right\}\right] \quad \ldots\ldots\ldots (7.3)$$

More simply, when $x_1=0$ and $C_1=C_{in}$, Eq. 7.3 becomes:

$$\frac{C}{C_{in}} = EXP\left[\frac{-\lambda_o}{a(1-n)}\left\{(1+ax)^{1-n}-1\right\}\right] \quad \ldots\ldots\ldots\ldots (7.4)$$

The model in Eq. 7.4 incorporates the most significant parameters that usually affect the filtration process, such as filter depth, filter material size, diameter of particles being filtered and the approach velocity. The model does not show the effect of accumulation of solids.

However, it was shown experimentally that the effect of accumulation of solids is to improve removal in the first stages due to increase in surface area. Deterioration will start when accumulation reaches a value such that the pores become smaller, and the shear force due to increase of interstitial velocity becomes more important than the increase in surface area (Ives and Sholji, 1965; Ives, 1969, 1970, 1975a; Amirtharajah and Wetstein, 1980).

The results of this research support the finding that both surface area of the rock and interstitial velocity are important and, in the beginning, the effect of surface area is more important. The decrease in TSS percentage removal with increase in hydraulic loading was slightly steeper for the filter with 10 cm rock

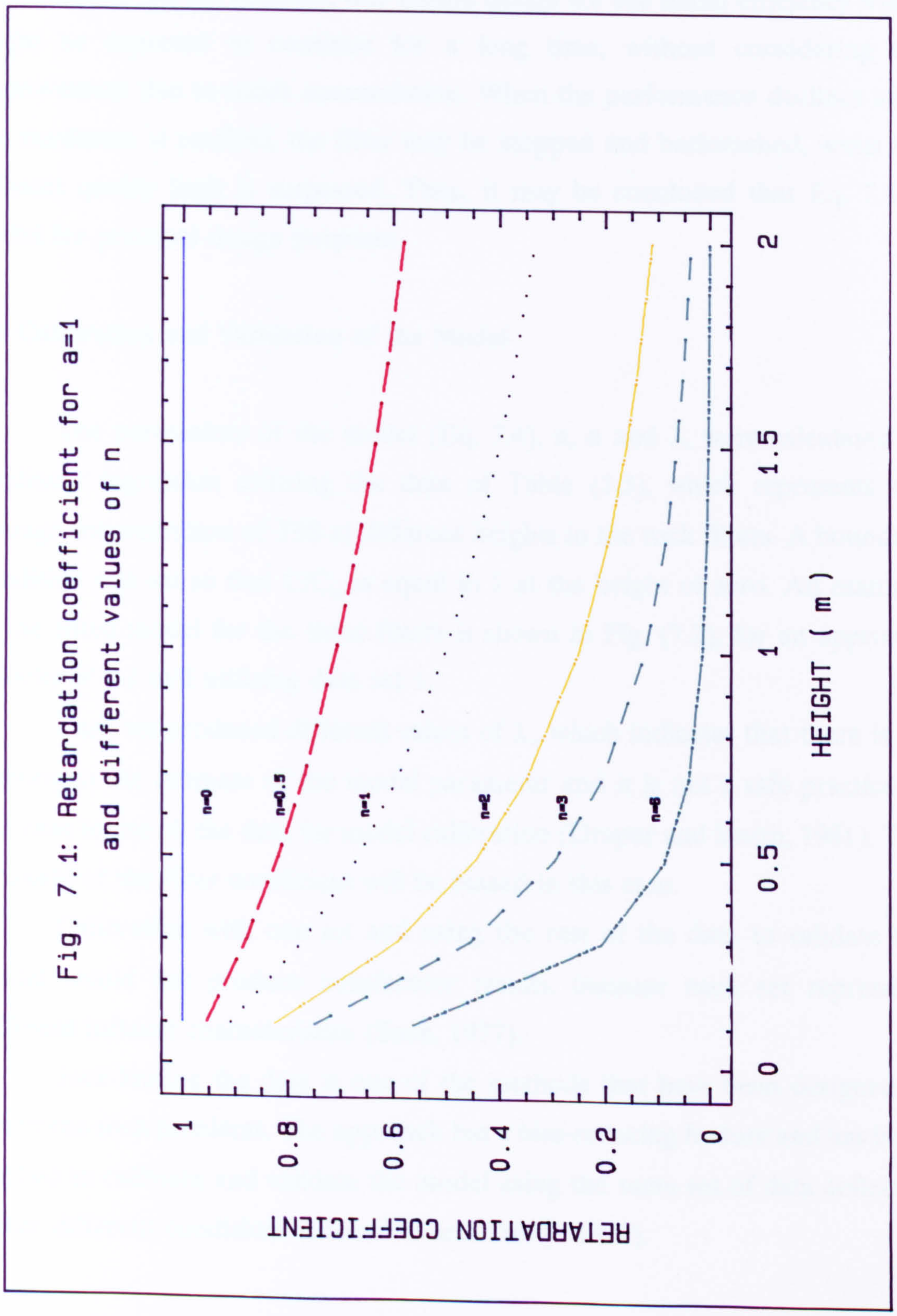


Fig. 7.1: Retardation coefficient for a=1 and different values of n

size than for that with 1 cm rock size (Fig. 6.1). This means that increasing v_i had less effect in the filter with small rock and larger available surface area.

Based on the above discussion, the design of a filter should not be based on the condition when removal is at its best, i.e when accumulation enhances performance. On the contrary, one should design for the initial efficiency which might be expected to continue for a long time, without considering the improvement due to solids accumulation. When the performance declines after the maximum is reached, the filter may be stopped and backwashed, when the effluent quality limit is surpassed. Thus, it may be concluded that Eq. 7.4 is useful for practical design purposes.

7.3 Calibration and Validation of the Model

The parameters of the model (Eq. 7.4), **a**, **n** and λ_o were calculated by nonlinear regression utilizing the data of Table (5.3), which represents the average concentration of TSS at different heights in the rock filters. A boundary condition was set so that C/C_{in} is equal to 1 at the height of zero. An example of the fitted model for the three filters is shown in Fig. (7.2), for an approach velocity of 0.2 m/d utilizing data set 1.

Each set produced different values of λ_o, which indicates that there is no stability in the estimate of the model parameter and it is not a safe practice in this case to use all the data for model calibration (Draper and Smith, 1981). The estimate of the filter coefficient will be biased in this case.

Calibration with one set and using the rest of the data to validate the model would not produce satisfactory results, because each set represents different influent characteristics (Snee, 1977).

Jack-knifing the data is one of the methods that have been designed to overcome such problems. The approach has a bias-reducing feature and has been applied to calibrate and validate the model using the same set of data collected under different conditions (Mosteller and Tukey, 1977).

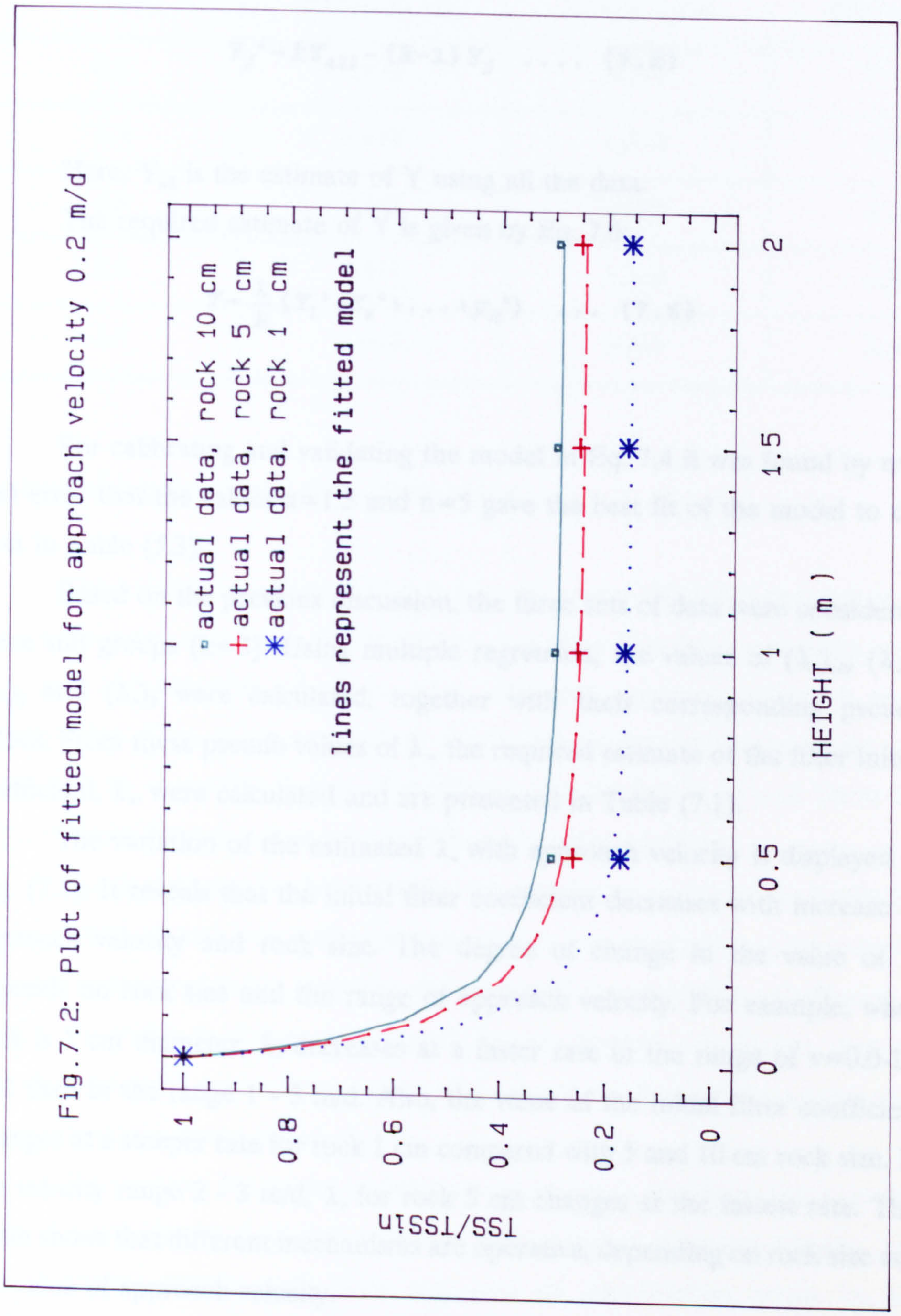


Fig.7.2: Plot of fitted model for approach velocity 0.2 m/d
lines represent the fitted model
actual data, rock 10 cm
actual data, rock 5 cm
actual data, rock 1 cm
HEIGHT (m)
TSS/TSSin
1
0.8
0.6
0.4
0.2
0
0.5
1.5
2

The jack-knife approach comprises dividing the data into k subgroups and calculating the required parameter, Y_j, by omitting the jth group. Then a pseudo value, Y_j^*, is calculated in Eq. 7.5:

$$Y_j^* = kY_{all} - (k-1)Y_j \quad .... \quad (7.5)$$

Here, Y_{all} is the estimate of Y using all the data.

The required estimate of Y is given by Eq. 7.6:

$$Y = \frac{1}{k}(Y_1^* + Y_2^* + ... + y_k^*) \quad ... \quad (7.6)$$

For calibrating and validating the model in Eq. 7.4 it was found by trial and error that the values a=1.5 and n=5 gave the best fit of the model to the data in Table (5.3).

Based on the previous discussion, the three sets of data were considered three sub groups (k=3). Using multiple regression, the values of $(\lambda_\bullet)_{all}$, $(\lambda_\bullet)_1$, $(\lambda_\bullet)_2$ and $(\lambda_\bullet)_3$ were calculated, together with their corresponding pseudo values. From these pseudo values of $\lambda_\bullet$, the required estimate of the filter initial coefficient, $\lambda_\bullet$, were calculated and are presented in Table (7.1).

The variation of the estimated $\lambda_\bullet$ with approach velocity is displayed in Fig. (7.3). It reveals that the initial filter coefficient decreases with increase of approach velocity and rock size. The degree of change in the value of $\lambda_\bullet$ depends on rock size and the range of approach velocity. For example, when rock is 1 cm diameter, $\lambda_\bullet$ decreases at a faster rate in the range of v=0.0-1.0 m/d than in the range 1 - 3 m/d. Also, the value of the initial filter coefficient changes at a steeper rate for rock 1 cm compared with 5 and 10 cm rock size. In the velocity range 2 - 3 m/d, $\lambda_\bullet$ for rock 5 cm changes at the fastest rate. This again shows that different mechanisms are operative, depending on rock size and the value of approach velocity.

Fig. (7.4) shows the variation of $\lambda_\bullet$ with time at v=0.2 m/d using the data of set 3 (see Appendix-A). The effect of time on $\lambda_\bullet$ is not clear but is assumed to be insignificant due to the relatively small time over which $\lambda_\bullet$ was calculated.

Table 7.1: Estimated initial filter coefficient, λ_o, for different approach velocities and rock sizes, m^{-1}.

v (m/d)	ROCK 10 cm	ROCK 5 cm	ROCK 1 cm
0.2	5.575	6.571	9.772
1.0	4.612	6.334	6.737
2.0	3.505	4.030	5.641
3.0	2.971	3.255	5.560
4.0	1.634	2.213	3.944

7.4 Implementation of the Rock Filter Model

The parameters of the model, a, n and λ_o, were calibrated and validated for a wide range of influent characteristics, including temperature and the type, size and concentration of algae. Also, different approach velocities were incorporated. Thus, it is reasonable to accept that the model may be taken as a basis, with an acceptable degree of confidence, for the design of rock filters for algae removal from facultative pond effluent in other locations.

The model was also developed for three rock sizes and 5 levels of velocity. These ranges of rock size and approach velocity are expected to represent the acceptable practical levels. However, the change of the initial filter coefficient λ_o (Fig. 7.3) seems to follow a systematic trend. Hence, this Figure may be extrapolated, with caution, for values of λ_o at approach velocities and/or rock sizes outside the range tested.

The engineer has to choose between a larger rock size filter with a smaller approach velocity, or a smaller rock size with a higher approach velocity. The availability of certain rock sizes and cost of construction might be the determining factors in any location. However, if high effluent quality is required, then the designer may be forced to choose a low approach velocity and small rock size.

Fig.7.3: Variation of initial filter coefficient with approach velocity

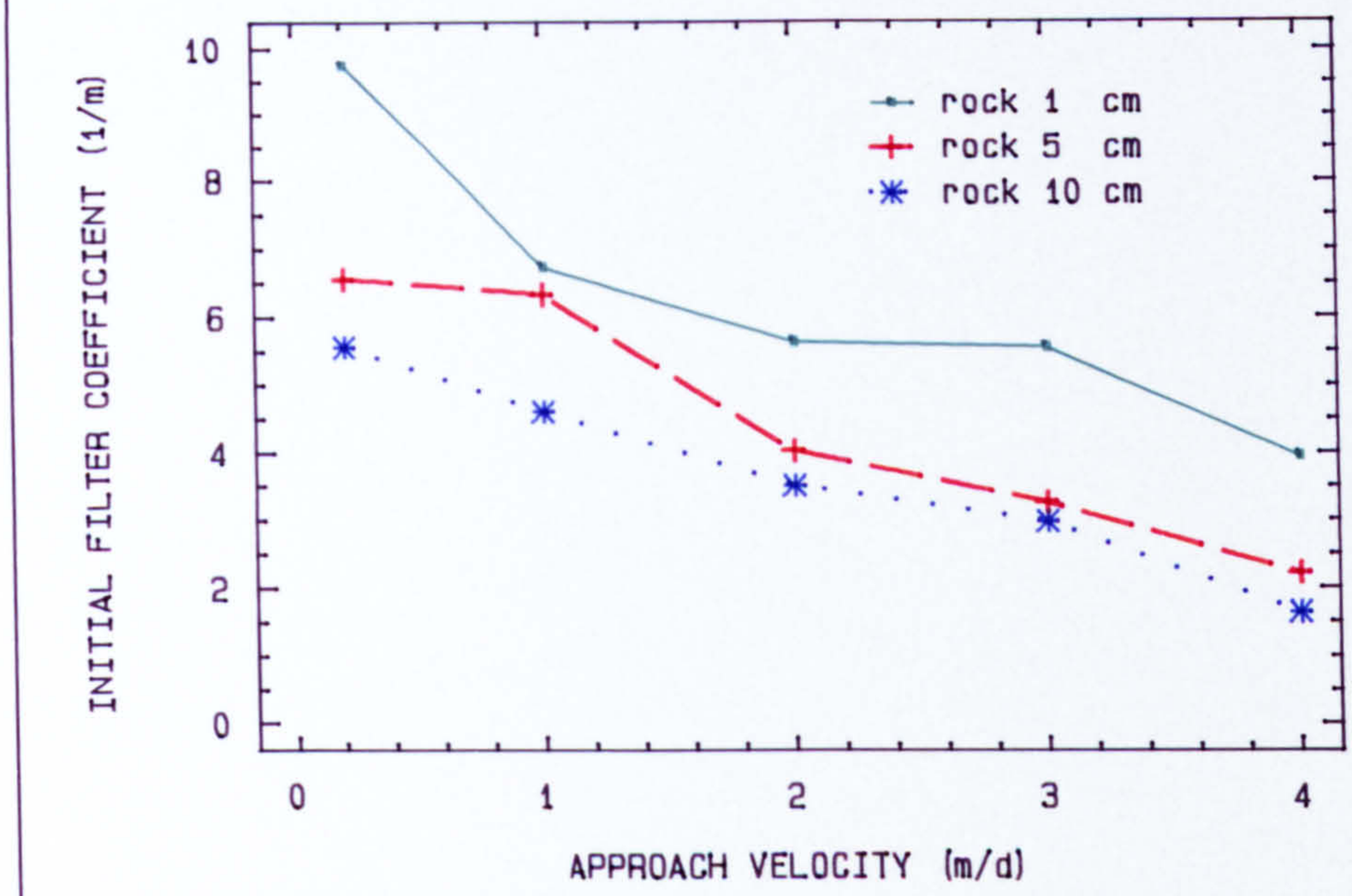
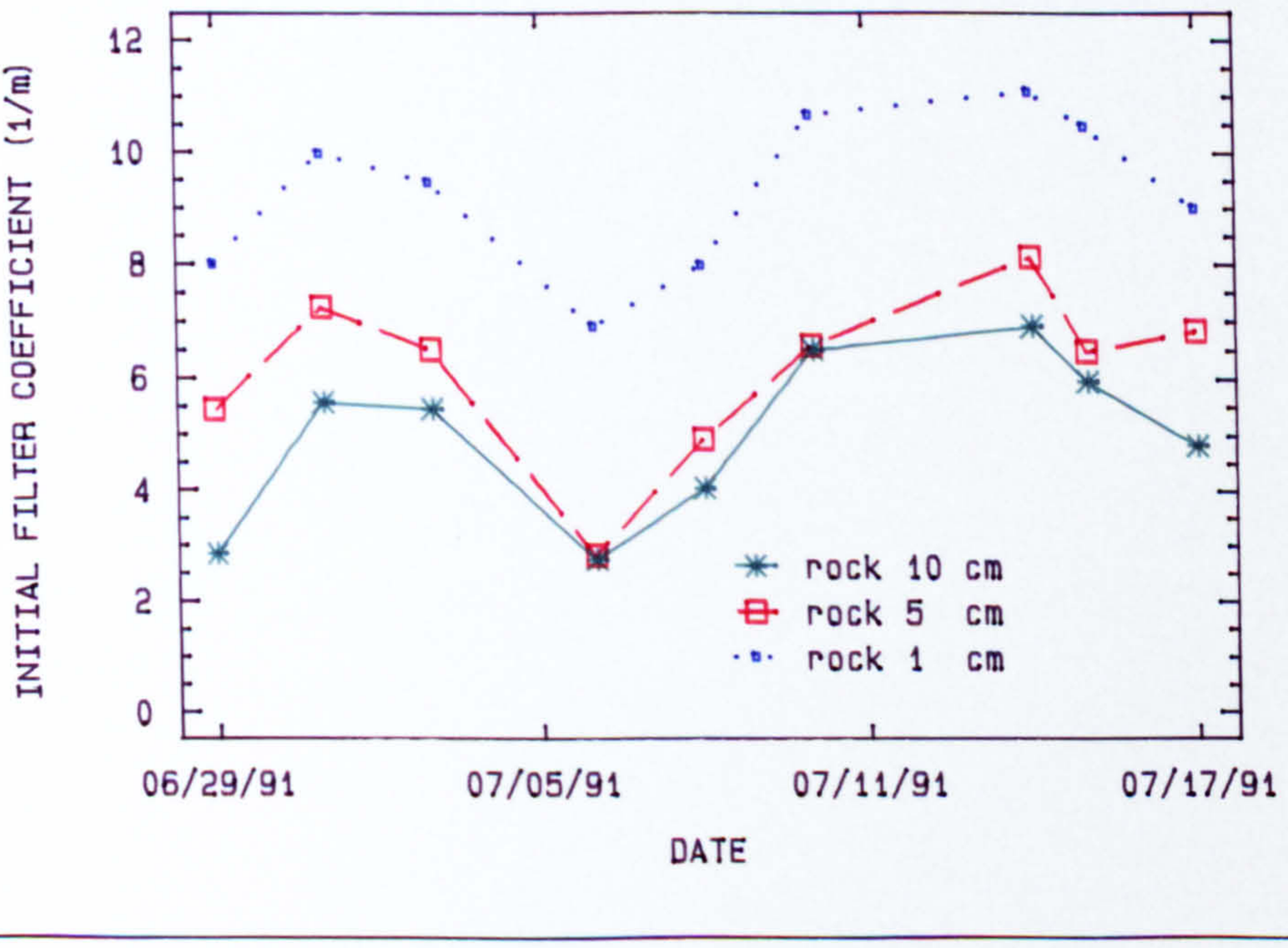


Fig.7.4: Variation of initial filter coefficient with time at v=0.2 m/d in experimental set 3.



Once the influent concentration is known, and depending on the required effluent quality and the practical approach velocity, the engineer will choose the most appropriate rock size. The corresponding filter coefficient from Table (7.1) or Fig.(7.3) can be applied to calculate the estimated effluent TSS concentration.

Trial and error may be adopted to determine the best combination of rock size and approach velocity.

CHAPTER 8

Al-Samra Ponds and King Talal Reservoir in Jordan

It was mentioned in Chapter 1 that the effluent of Al-Samra ponds has a TSS concentration far exceeding the limits set by the Jordanian Revised Standards. Also, it was mentioned that the effluent of these ponds has been viewed by some workers as the cause of eutrophication in King Talal Reservoir.

This Chapter is intended to give an example of the application of the design process for an upflow rock filter for removal of suspended solids from Al-Samra effluent. Also, a mathematical model will be developed for the reservoir behind King Talal Dam so as to help in the decision making process for control of eutrophication in the reservoir and to find the role of Al-Samra ponds' effluent in this process.

8.1 Removal of Solids from Al-Samra Ponds' Effluent

The monthly average TSS concentration in Al-Samra effluent is in the range of 100-240 mg/l (RSS, 1991). The Jordanian standards require that this concentration be reduced to a level not greater than 50 mg/l to meet the limits for disposal of effluent in wadis and reservoirs.

The average discharge of Al-Samra ponds in 1991 was 80,000 m^3/d (55 m^3/minute).

To reduce 240 mg/l of TSS in the effluent to less than 50 mg/l requires that the efficiency of removal of the filter should be at least 80%. In order to achieve such a degree of removal, the preliminary choice is a rock filter with small size packing material(1 cm) and approach velocity of 0.2 m/d or less (Fig. 6.1). Unfortunately, such small velocities will not minimize the filter area required.

At an approach velocity of 0.2 m/d and a rock size of 1 cm, the initial filter coefficient will be 9.8 (Table 7.1, Fig. 7.3). With C_{in} = 240 mg/l, n = 5, a = 1.5 and 1m filter height, Eq. 7.4 yields an effluent concentration C = 49mg/l.

The area required will be 40 ha. Such an area is very large and constitutes 22% of the water area of 183 ha at mean depth of the existing ponds at Al-Samra.

8.2 Eutrophication in King Talal Reservoir

King Talal Reservoir (KTR) is an earth fill dam constructed across the Zarqa River. It was completed in 1977 with a capacity of 50 million cubic metres (MCM) increased to 86 MCM in 1986. The water captured behind the dam was intended to be used for irrigation downstream in the Jordan Valley, and for supply of drinking water to the capital, Amman (RSS, 1981-1988). The reservoir water has been used for irrigation but never for drinking purposes because of the heavy metals carried to the reservoir from industries located in the Zarqa River basin.

Since the effluent of Al-Samra ponds started to reach KTR in 1985, criticism has been laid on Al-Samra as the cause of eutrophication and deterioration of water quality in the reservoir (Salameh et al., 1987). However, no attempt has been made to construct a mathematical model that describes the eutrophic status of the reservoir and make a more reliable decision about the relationship between the reservoir eutrophication and Al-Samra effluent.

8.2.1 Control of Eutrophication

Eutrophication is the continuous feed of nutrients, N, P and CO_2, to bodies of water such that growth of aquatic life is proliferated. Eutrophication could be natural, taking place slowly over the years, or rapid, due to human activities which stimulate primarily the growth of algae (Likens, 1972; Palmer, 1980; Sterritt and Lester, 1988).

Eutrophication is characterized by less diversity and luxurious development of blooms of certain species of algae, such as blue green forms, accompanied by deterioration of water quality by lowering DO, producing odour and taste nuisance, and forming a suitable environment for fly and mosquito

breeding. A high rate of algal photosynthesis brings pH up to levels greater than 11, which causes fish kills. In addition, certain species of algae produce toxins that may threaten animal life or cause allergic reactions in humans upon contact (Bold and Wynne, 1985).

To control eutrophication, enrichment of the body of water with nutrients should be decreased or stopped. This raises three questions: the first concerning the sources that should be eliminated, the second about the limiting nutrient that should be restricted, and the third to what degree should nutrient inputs be reduced.

It is obvious that control of point sources such as effluents from wastewater treatment plants is more feasible than controlling dispersed sources of nutrients, such as those originating from agricultural rain shed areas.

The question of limiting nutrient in the eutrophication process has been a matter of controversy among researchers. The idea of limiting nutrient is usually based on the ratio of nutrients forming the algal cell. That element which exists in concentration short of this structural ratio forms the limiting nutrient. This is known as Liebig's law (Zajic, 1971). Some researchers found that carbon was the limiting element in growth of algae (King, 1970; Kerr et al., 1972; Simmonds, 1973). Others found N was limiting in the marine environment (Stumm, 1973; Goldman, 1976; Sterritt and Lester, 1988). Some found P was limiting (Powers et al., 1972; Stumm, 1973; Jordan and Bender, 1973). Maloney et al. (1972) studied nine Oregon lakes and found that algal growth rate was proportional to the concentration of phosphorus but no correlation was noticed with the concentration of N or C.

However, the concept of limiting nutrient is a simplification of the process because, although the growth of a single cell may be limited by a certain nutrient at a certain time, this is not the case for a complex ecosystem, and even one cell may be limited by more than one element at different times because of the requirements of different enzyme systems within a single cell (Zajic, 1971; Goldman, 1972). Besides, different structural formulae have been given to different species of algae. McCarty (1970) suggested the formula C_{106} H_{18}, N_{16} P, while the formula given to algae according to Oswald and Gotaas (1955) was

$C_{7.62}$ $H_{8.06}$ $O_{2.53}$ $N_{1.0}$. Hemens and Stander (1969) reported that P constitutes 1% of the dry weight of different species of algae, while N forms 7.5% in the *Cyanophytes* and 4.2% in the *Chlorophytes*. The ratio of C:N:P in algal cells is known to differ according to the environment (Lee, 1973) and it has been found that in many lakes sometimes N was limiting and on other occasions P was limiting in the same lake (Jenkins et al., 1973).

Also, it should be noted that the N:P ratio in the phytoplankton should not be considered as the ratio which determines the consumption and diminishing of nutrients in the media because nutrients have different levels of availability to algae, and are recycled at different rates. For example, most of the carbon is not available to algae and remineralization brings back P more rapidly than N in the media (Kerr et al., 1972; McCarthy, 1980).

In spite of such disagreements about the limiting nutrient concept, it has been found that control of phosphorus is the most feasible because CO_2 can be replenished through equilibrium with the atmosphere, and deficiency of N or C encourages the growth of blue green algae, which can fix nitrogen from the atmosphere (Buzzell and Swayer, 1967; Goodman and Mikkelson, 1970; King, 1972; Morton, 1972; Snoyink and Jenkins, 1980; Boney,1989).

Lee (1973) considered phosphorus as the key element for control of eutrophication. In the beginning, the key element might not be limiting the growth but it will become limiting through the control process.

Edmondson (1972) found that a reduction of P input to lake Washington controlled phytoplankton production and increased transparency. Ferguson and King (1977) reported that eutrophication control was accomplished through phosphate removal from wastewater in several regions in the U.S.A. Bernardt et al. (1971) found that reduction of the P level would prevent algal blooming, in spite of a high nitrogen load. Although small amounts of phosphorus are needed to stimulate growth of algae, it has been found that diversion of wastewater from lower Madison, Wisconsin lakes improved their quality and diversified algal population in spite of not reaching the required low concentrations of P in the lakes (Lee, 1973). Wood and Gibson (1973) suggested that P loading should be

reduced to Lough Neagh in Northern Ireland, the largest lake in the British Isles, in order to control eutrophication.

Brezonik (1972) reported that for the purpose of preventing eutrophication in a water body, inorganic phosphorus should be less than 0.015 mg/l and the concentration of N 0.3 mg/l. He suggested a permissible loading of 0.28 g-P/m^2.yr and 1.9 g-N/m^2.yr and considered dangerous loadings above 0.49 g/m^2.yr and 3.4 g/m^2.yr of P and N. Vollenwider (1968) related permissible and dangerous loading rates of P and N to the depth of the lake (Table 8.1).

Sawyer and McCarty (1978) considered 0.005 mg/l the critical concentration of inorganic P for algal growth under Summer conditions. Algal blooms will occur when dissolved phosphorus exceeds this limit. Bernardt et al. (1971) suggested 0.01 mg/l-P concentration for control of eutrophication in Wahnbach Reservoir.

Table 8.1: Permissible P and N loadings to lakes

Source: Vollenwider (1968)

depth up to (m)	permissible loading (g/m^2.yr)		dangerous loading (g/m^2.yr)	
	N	P	N	P
5	1.0	0.07	2.0	0.13
10	1.5	0.10	3.0	0.20
50	4.0	0.25	8.0	0.50

8.3 Assumptions and Model Development

In the previous Section, it was shown that the key element for control of eutrophication is the reduction of P concentration. Total P concentration will be used as a measure of trophic status because it is assumed that all forms of phosphorus will become eventually available to algae for assimilation (Thomann and Mueller, 1987).

Fig. (8.1) illustrates all the sources and sinks of phosphorus to King Talal Reservoir. These sources and sinks are tabulated in Table (8.2). The terms are defined as follows :

Q_r = inflow from the main river (Zarqa), m^3/yr

Q_T = inflow from tributaries (Rmeimin), m^3/yr

Q = outflow, m^3/yr.

V = volume of the lake, m^3.

A_s = lake surface area, m^2.

P = precipitation directly on the lake, m/yr.

E_v = evaporation from the lake, m/yr.

S_D = rate of phosphorus release from sediment, $kg/m^3.yr$

C_{pr} = concentration of phosphorus in the precipitate, kg/m^3

C = concentration of phosphorus in the lake, kg/m^3

C_r = concentration of phosphorus in the main river, kg/m^3

C_T = concentration of phosphorus in the tributary, kg/m^3

S_F = dry fallout rate, $kg/m^2.yr$

Two equations can be written for the system in Fig. (8.1). Eq. 8.1 represents the balance of flows in and out of the system. Eq. 8.2 represents the mass rate of loading of phosphorus to the system, except for the release of sediments, resuspension of settled solids, release of the geological formation and decomposition of algal and bacterial cells.

$$Q = Q_r + Q_T + PA_s - E_v A_s \quad \text{..... (8.1)}$$

$$W(t) = Q_r C_r + Q_T C_T + PA_s C_{pr} + S_F A_s \quad \text{..... (8.2)}$$

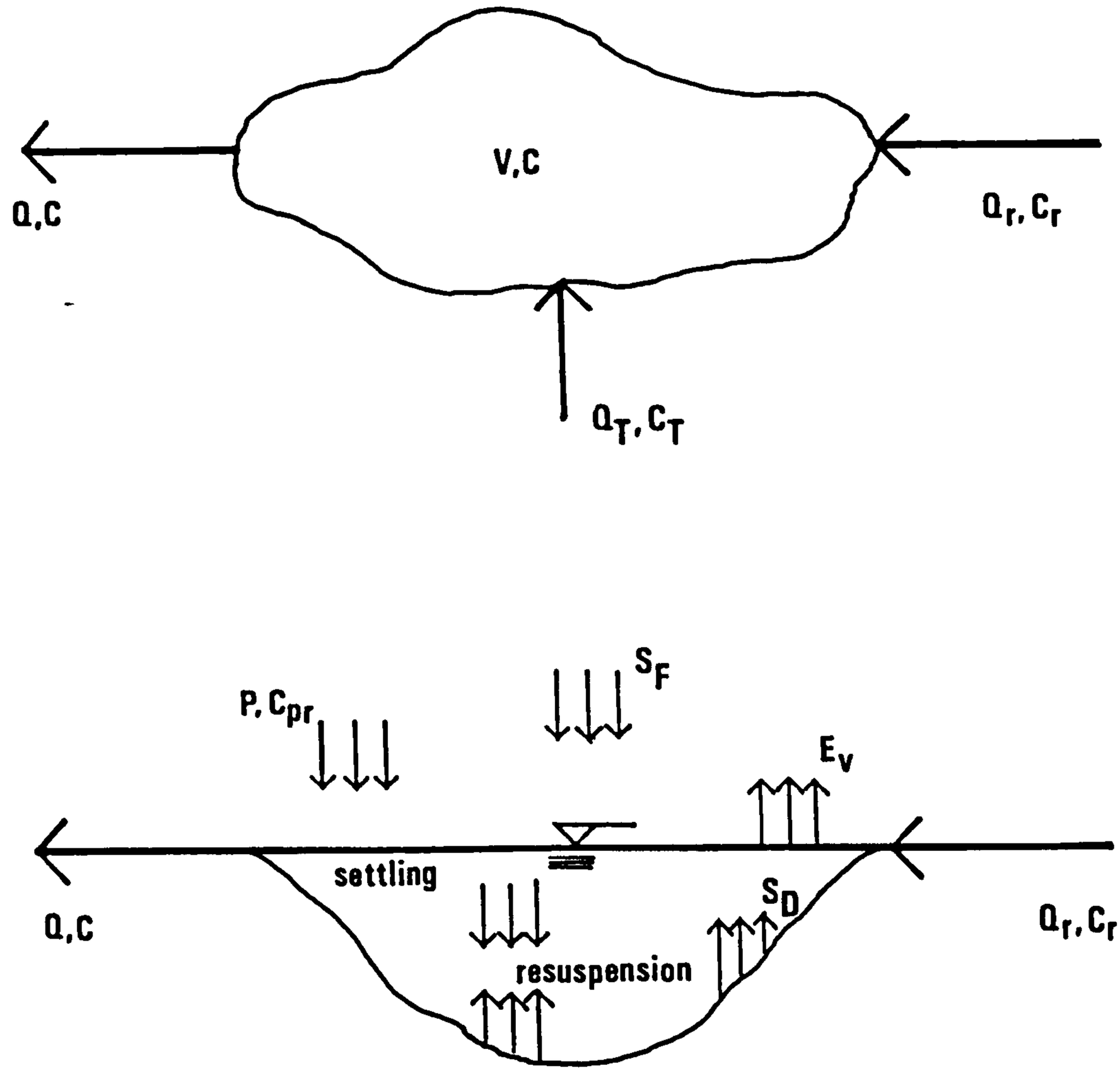


Fig.8.1a: Schematic representation of sources and sinks of phosphorus to KTR

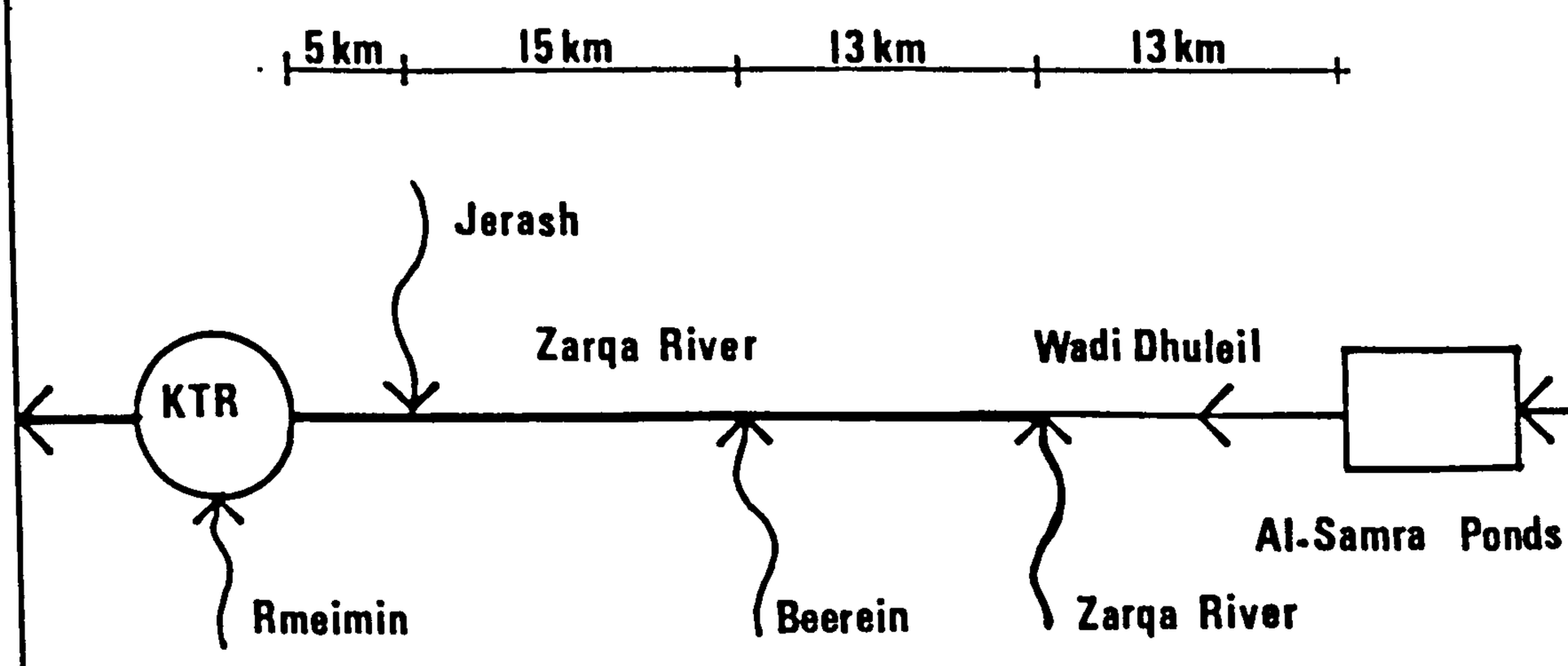


Fig.8.1b: Schematic diagram of the Zarqa River

Table (8.2): Sources and sinks of phosphorus to KTR

Sources of Phosphorus	Sinks of Phosphorus
(1) main river, Q_r C_r	(a) effluent, QC
(2) tributaries, Q_T C_T	(b) settling of solids
(3) sediment release, S_D V	(c) synthesis of algal cells and bacteria
(4) resuspension of settled solids	
(5) release from the geological formation	
(6)precipitayion, PA_sC_{pr}	
(7) dry fallout, S_FA_s	
(8) decomposition of algal cells and bacteria	

It is assumed that the reservoir behind the dam is a completely-mixed flow reactor. This assumption is justified if the time scale is long (e. g. on a yearly basis) because seasonal overturn, in addition to wind, may result in a completely-mixed reservoir (Thomann and Mueller, 1987).

The accumulation of total phosphorus concentration in the water of the lake may be given by the continuity equation (Eq. 8.3) (Chapra and Reckhow, 1983):

$$\frac{d}{dt}(VC) = W(t) - QC - KVC \quad \ldots\ldots (8.3)$$

$$V\frac{dC}{dt} + C\frac{dV}{dt} = W(t) - QC - KVC \quad \ldots (8.4)$$

Assuming a constant yearly volume of the lake, results in:

$$V\frac{dC}{dt} = W(t) - QC - KVC \quad \ldots\ldots (8.5)$$

In Eq. 8.5 it is assumed that phosphorus is taken out of solution by first order kinetics with a rate constant, K (day^{-1}). The rate constant, K, is a lump sum parameter incorporating, net settling and resuspension of P in particulate form, and release of phosphorus at the bottom of the lake from sediments and geological formations.

Synthesis of algal and bacterial cells and their decomposition is included in Eq. 8.5 by considering total phosphorus rather than dissolved phosphorus concentration.

The solution of Eq. 8.5 is composed of two parts . The first is a complementary solution, C_c, when W(t)=0. The second deals with the case when W(t) is a certain function and the solution is called the particular solution, C_p (Chapra and Reckhow, 1983; Thomann and Mueller, 1987).

The complementary solution of Eq. 8.5 is given by Eq. 8.6:

$$C_c = C_o e^{-\alpha t} \quad \ldots\ldots (8.6)$$

where, $\alpha = Q/V + K$

The particular solution may be found by multiplying Eq. 8.5 with the integration factor, $e^{\alpha t}$.

$$e^{\alpha t}.\frac{dC}{dt} + e^{\alpha t}.\alpha C = e^{\alpha t}.\frac{W(t)}{V} \quad \ldots\ldots (8.7)$$

$$\frac{d}{dt}(e^{\alpha t}C) = e^{\alpha t}.\frac{W(t)}{V} \quad \ldots\ldots (8.8)$$

Introducing a dummy variable τ, and integrating from zero to t, Eq. 8.8 becomes:

$$\int_0^t \frac{d}{d\tau}(e^{\alpha\tau}C)\,.\,d\tau - \frac{1}{V}\int_0^t e^{\alpha\tau}W(\tau)\,.\,d\tau \ldots\ldots\ldots (8.9)$$

$$e^{\alpha t}C - C_\bullet - \frac{1}{V}\int_0^t e^{\alpha\tau}W(\tau)\,.\,d\tau \ldots\ldots\ldots (8.10)$$

Dividing by $e^{\alpha t}$, gives:

$$C(t) - C_\bullet e^{-\alpha t} + \frac{1}{V}\int_0^t e^{-\alpha(t-\tau)}W(\tau)\,.\,d\tau \ldots\ldots (8.11)$$

The first part of the right hand side of Eq. 8.11 is the complementary solution given in Eq. 8.6, and the second part on the same side forms the particular solution, C_p.

8.4 Calibration of the Model

Based on the data given by RSS (1981-1988) the average discharge from the dam, Q, was 58.9136 MCM/yr. The five year average yearly volume of water behind the dam was 26.848 MCM. Total phosphorus concentration in the centre of the lake is given in Table (8.3)

Table 8.3: Total phosphorus concentration in KTR

Source: RSS (1981-1988) and RSS (1991)

year	81	82	83	84	85	86	87	88	89
TP mg/l	.1	.1	.17	.31	.20	.88	1.1	1.4	2.3

Using the data given in the above reference, an analysis was carried out for total phosphorus yearly loading in kg/year, which was found to increase exponentially according to Eq. 8.12.

$$W(t) = be^{\beta t} \quad \text{..... (8.12)}$$

Where, b= 18958, β= 0.36 and t is the time in years.

In Eq. 8.12, W(t) represents phosphorus loading through the tributary and the main river. Dry fallout and concentration of phosphorus in the precipitate were assumed equal to zero.

When the load input is in the form of Eq. 8.12, the particular solution of Eq. 8.11 becomes:

$$C_p = \frac{1}{V}\int_0^t e^{-\alpha(t-\tau)} . b\, e^{\beta\tau} . d\tau \quad \text{.....(8.13)}$$

$$C_p = \frac{b}{V} . e^{-\alpha t}\int_0^t e^{(\alpha+\beta)\tau} . d\tau \quad \text{.....(8.14)}$$

$$C_p = \frac{b}{V} . e^{-\alpha t} . \frac{1}{(\alpha+\beta)} [e^{(\alpha+\beta)t} - 1] \quad \text{.....(8.15)}$$

$$C_p = \frac{b}{(\alpha+\beta)\,V} . [e^{\beta t} - e^{-\alpha t}] \quad \text{.....(8.16)}$$

There are no data available for the concentration of phosphorus in the first two years of the life of the reservoir. So, assuming that the concentration of phosphorus at the end of (1980) is the initial concentration, C_o = 0.05 mg/l, then

Eq. 8.11 can be used to determine α with the particular solution as given in Eq. 8.16, utilizing the values given previously for b, β, V, and the data of Table (8.2).

Using nonlinear regression, it was found that α= 7.907. Hence, K= 5.713 yr^{-1} or 0.0156 day^{-1}.

The plot of the fitted model with the actual concentration is show in Fig. (8.2).

8.5 Input Phosphorus Reduction

It is clear from Eq. 8.16 and Fig. (8.2) that the situation will further deteriorate if phosphorus loading keeps increasing at the same rate. The question raised here is about the degree of reduction of phosphorus required in order that recovery takes place in the reservoir and an oligotrophic state is reached. Assuming that the critical phosphorus concentration is 0.01 mg/l, and the new phosphorus loading should be reduced to a constant step load, $\bar{W}$, then it can be shown that the particular response of Eq. 8.5 is given by Eq. 8.17:

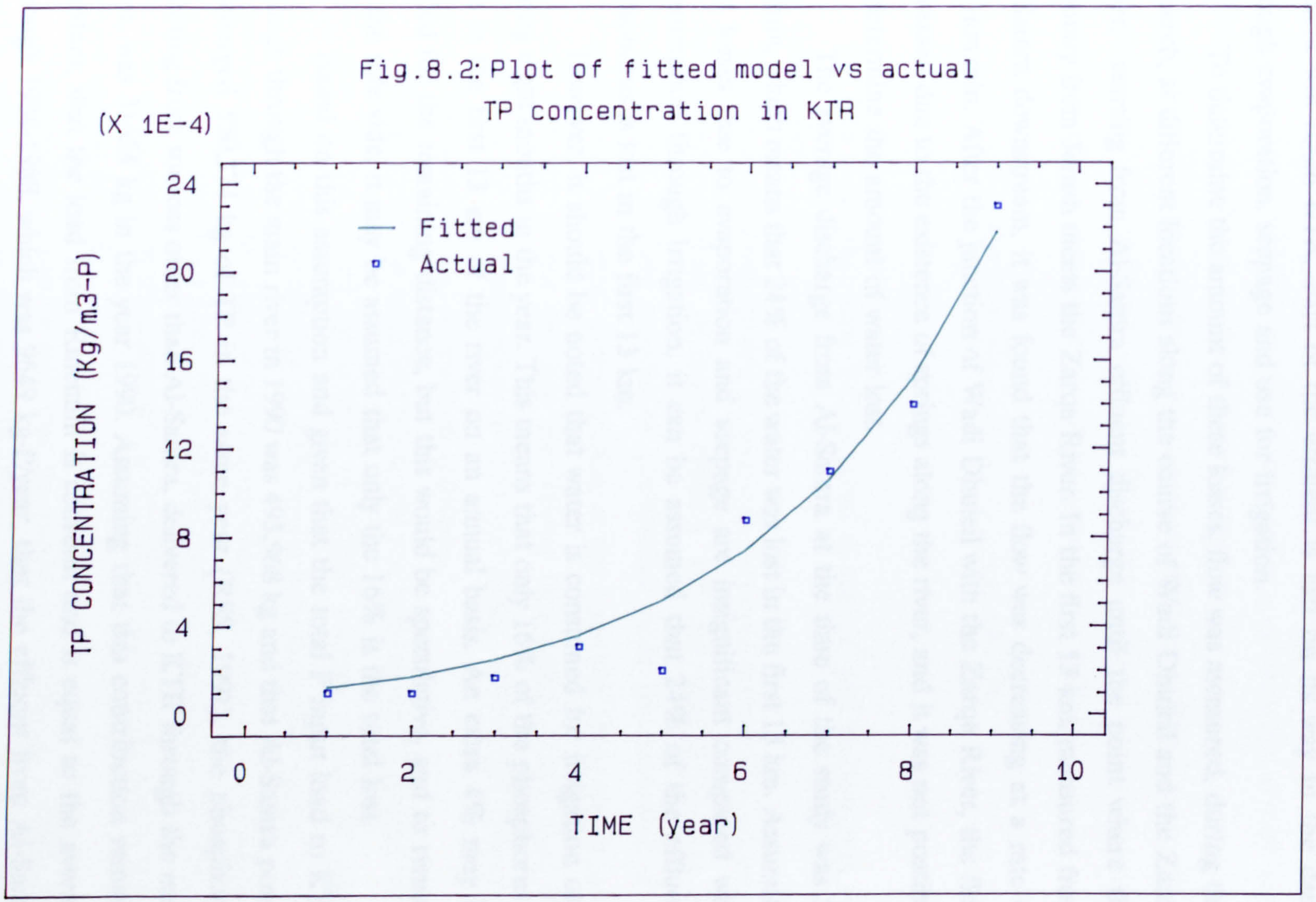
$$C_p = \frac{\bar{W}}{\alpha V}.[1-e^{-\alpha t}] \quad .....(8.17)$$

Management of diffuse sources of phosphorus is complex and it is easier to control point sources. Fig. (8.1) depicts different sources of phosphorus to the lake. The three point sources are Al-Samra ponds, Rmeimin tributary and Jerash wastewater treatment plant. However, effluent of the latter is completely used for irrigation and does not reach the Zarqa River. The contribution of Rmeimin was only 3% of the average main river loading from 1984-to-1990. This leaves Al-Samra as the potential source that could be controlled.

The problem now is the determination of $\bar{W}$, if phosphorus is partly removed by banning of phosphorus-based detergents, for example, or by treatment of Al-Samra effluent.

It is not plausible to assume that complete removal of P from Al-Samra effluent would return phosphorus loading levels to those before the operation of the ponds. This is because many activities, especially farming, have been growing

Fig.8.2: Plot of fitted model vs actual
TP concentration in KTR



extensively around Wadi Dhuleil and the Zarqa River basin, since 1985 when Al-Samra effluent started to flow in the Wadi. Thus, new phosphorus sources have been added to the system.

Also, it should be noted that not all phosphorus released with Al-Samra effluent will reach KTR. Part of the effluent is lost on its way to the dam, through evaporation, seepage and use for irrigation.

To determine the amount of these losses, flow was measured, during this research, at different locations along the course of Wadi Dhuleil and the Zarqa River, starting from Al-Samra effluent discharge until the point where the tributary from Jerash meets the Zarqa River. In the first 13 km, measured from Al-Samra downstream, it was found that the flow was decreasing at a rate of 1m³/km.min. After the junction of Wadi Dhuleil with the Zarqa River, the flow increased due to the existence of springs along the river, and it was not possible to determine the amount of water lost.

The average discharge from Al-Samra at the time of the study was 55 m³/min, which means that 24% of the water was lost in the first 13 km. Assuming that losses due to evaporation and seepage are insignificant compared with consumption through irrigation, it can be assumed that 24% of the effluent phosphorus is lost in the first 13 km.

However, it should be noted that water is consumed for irrigation only during eight months in the year. This means that only 16% of the phosphorus is lost in the first 13 km of the river on an annual basis. An extra 4% may be added for the remaining distance, but this would be speculative, and to remain on the safe side, it may be assumed that only the 16% is the total loss.

Based on this assumption and given that the total P input load to KTR through through the main river in 1990 was 493,968 kg and that Al-Samra ponds discharged 550,374 kg of TP in the same year (RSS, 1991), the phosphorus resulting from sources other than Al-Samra, delivered to KTR through the main river, was 31654 kg in the year 1990. Assuming that this contribution remains constant, that the load from Rmeimin is constant and is equal to the average through 1984-1988, which was 9649 kg-P/year, that the effluent from Al-Samra remains constant at approximately 60 m³/min with effluent TP concentration of

6.5 mg/l after banning phosphorus-based detergents (see Table 5.5), then the constant contribution of Al-Samra will be 51.333 m^3/min [60-(13*8/12)] or 175375 kg-P/yr. Hence $\bar{W}$ is 216678 kg-P/yr [31654+9649+175375].

Fig. (8.3) shows the response of the reservoir to the new loading conditions and reveals that, after two years, the concentration will reach a steady state of about 1mg/l, which is far above the critical concentration of 0.01 mg/l assumed earlier and based on the reviewed literature.

Abatement of phosphorus completely from Al-Samra effluent means that the loading to the reservoir will be $\bar{W}$=41303 kg/yr. The steady state concentration after a very long time will be given by Eq. 8.18.

$$\bar{C} = C_o e^{-\alpha t} + \frac{\bar{W}}{\alpha V} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8.18)$$

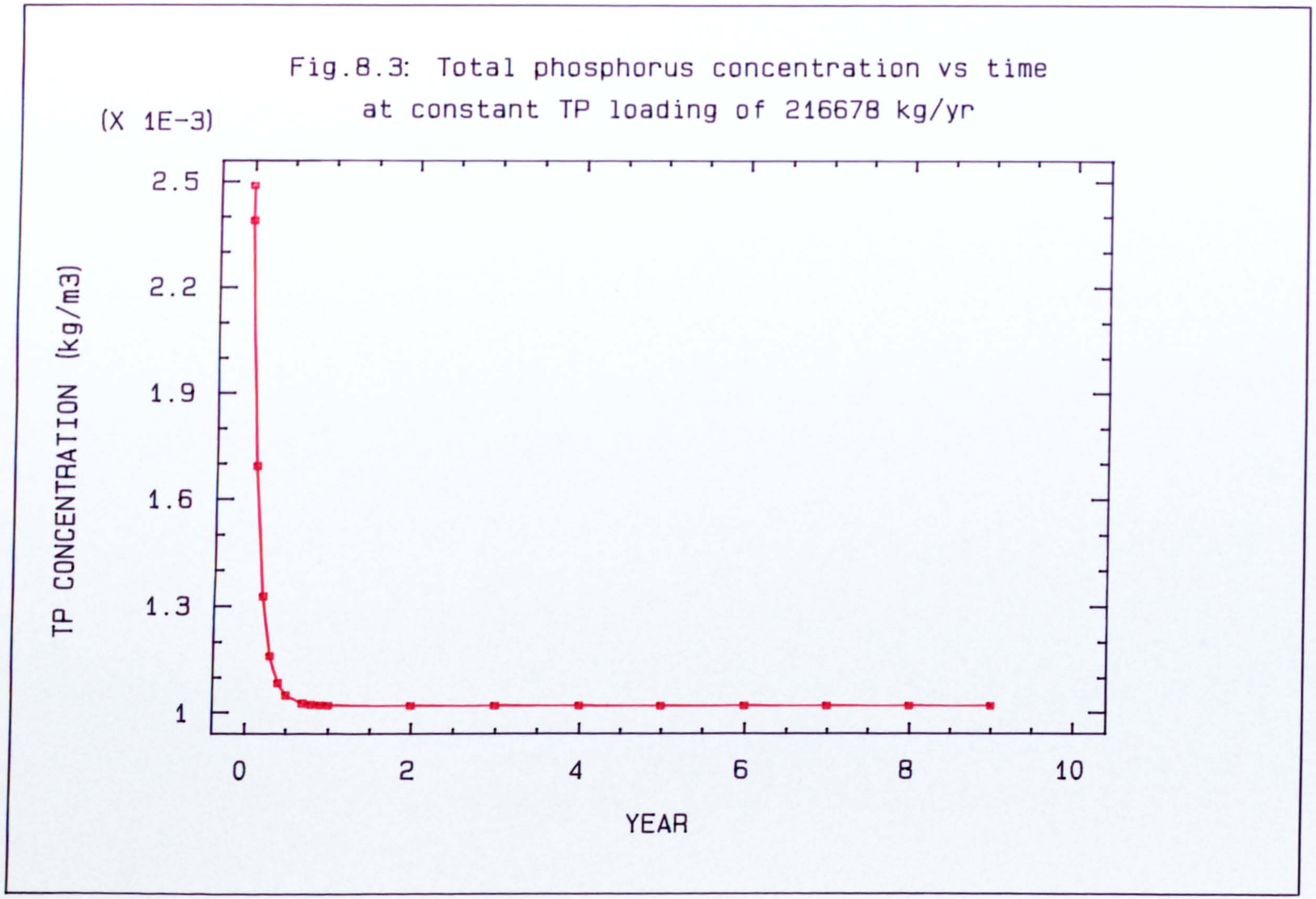
Eq. 8.18 shows that even if C_o is taken as zero, $\bar{C}$ will be 0.19 mg/l for $\bar{W}$= 41303 kg/yr.

Thus, it is not unreasonable to conclude that even complete elimination of total phosphorus from Al-Samra effluent will not take KTR into an oligotrophic state. Diffuse sources of phosphorus must be controlled if it becomes essential to control eutrophication in the reservoir. However, it must be noted that this is not an easy task taking into consideration that the catchment area for the Zarqa River is 3157 km^2 (RSS, 1981-1988). The area abounds with industrial and agricultural activities.

It should be kept in mind that the model was calibrated with limited data. Application of the model for control of diffuse sources should be taken cautiously. Additional monitoring of KTR would help in further tuning of the parameters of the model. Also, it is recommended that more a comprehensive study should be carried out along Wadi Dhuleil and the Zarqa River in order to determine more precisely the contribution of different sources of phosphorus to the load applied to KTR.

Nevertheless, it can be said that the model is reasonably credible and could be applied, with an acceptable level of confidence, to the decision

Fig.8.3: Total phosphorus concentration vs time
at constant TP loading of 216678 kg/yr



regarding treatment of Al-Samra effluent for phosphorus removal. This is because even upon elimination of phosphorus from Al-Samra effluent, the model predicted the steady state concentration in the lake at a value very much higher than is required to achieve an oligotrophic state.

CHAPTER 9

Conclusions and Recommendations

This research has revealed the effects of different parameters on the performance of an upflow rock filter for algae removal from facultative pond effluent. Also it has helped to identify more precisely the mechanisms that govern the process of removal of algae by upflow rock filtration. A general procedure was developed for design of these filters based on the results of this research.

The findings of this research demonstrate that removal of algae by coagulation of the facultative pond effluent, could be made a low cost process. This could be achieved by decreasing the dose of the coagulant or by employing primary coagulants that cost less.

More specific conclusions and recommendations which may be drawn out of this research are summarized in the following sections.

9.1 Conclusions

1. It was shown that upflow rock filtration is a potential feasible process for removal of algae from facultative pond effluent. Removal up to 80% of total suspended solids could be achieved, depending on the operation variables of the filter.

2. A more definite relationship was found between the ability of the rock filter to remove TSS and operating variables, such as approach velocity, diameter of the rock, concentration of suspended solids in the influent, and height of the filter. It was found that TSS removal decreased with increase of approach velocity and increase of rock size. Increase in influent suspended solids concentration generally increased TSS removal percentage. However, the effect of the concentration was found to be interrelated with both rock size and the level of approach velocity. It was found that removal increased with filter height.

However, a significant proportion of removal took place in the first half metre of the filter height.

3. Contrary to previous assumptions by other workers that gravity settling was the only mechanism responsible for algae removal in rock filters, the analysis of the results of this work revealed that other mechanisms contribute to the process of algae removal in the pores of an upflow rock filter. On average, hydrodynamic action was found to be the second most important mechanism after settling. In fact, hydrodynamic action may become more significant in the pores of small rock filters, where settling is hindered by the higher interstitial velocity.

Interception was found to be less important and diffusion was found almost insignificant.

Orthokinetic flocculation and autoflocculation are thought to play a major role in removal of algae.

4. A model was constructed to describe the relationship between the performance of the rock filter and operating variables of the filter. The model was formulated as follows:

$$\frac{C}{C_{in}} = \exp\left[\frac{-\lambda_o}{a(1-n)}\{(1+ax)^{1-n}-1\}\right]$$

The influence of operating variables, such as influent concentration of suspended solids and height of filter, are explicitly shown in the model. The effect of variables such as approach velocity and diameter of the rock is reflected in the value of initial filter coefficient, λ_o.

Furthermore, the value of λ_o reflects the effect of the variation of algal cell characteristics in terms of species, diameter, shape, and specific gravity of the cell. However, it must be kept in mind that λ_o was averaged over a relatively small time scale and this average value was assumed to represent the initial filter coefficient.

5. It was shown in this research that the design of rock filter on the basis of volumetric hydraulic loading ($m^3/m^3.d$), which has been widely used until now for horizontal flow rock filters, is invalid for upflow rock filters. It was shown that not all of the filter depth is actually taking a significant role in the process of removal of algae. Hence, approach velocity should be adopted for sizing the upflow rock filter instead of volumetric hydraulic loading.

6. Influent total COD and total BOD_5 were decreased by upflow rock filtration. Most of the time, removal of soluble BOD and COD was achieved by these filters .

Removal of ammonia was effected by the filters most of the time. However, occasional increase in ammonia concentration was noticed in the effluent over that of the influent. Increase of TKN was less frequent.

Total and soluble phosphorus were often reduced by the filters but an increase was noticed sometimes.

7. Optimum alum dose for removal of algae was found to be 450 mg/l for 40 minutes settling. It was shown that increase of sedimentation time could reduce the required alum dose. However, the effect of sedimentation time on alum dose was only noticed for low doses up to 80 mg/l. The effect was more pronounced when the dose was less than 40 mg/l.

8. Addition of kaolinite or bentonite as a coagulant aid, resulted in the formation of better settleable flocs. Better removal of turbidity was also achieved. The use of 250 mg/l of bentonite with 80 mg/l of alum achieved 88% removal of turbidity, a degree of removal which was achieved at 160 mg/l of alum alone.

Addition of these coagulant aids made more significant the effect of settling time on the degree of removal.

9. In this research, it was shown that kaolinite and bentonite were successful as primary coagulants for algae removal from facultative pond effluent. Generally, bentonite exhibited better ability for removal of turbidity than kaolinite.

Up to 60% removal of turbidity was be obtained by 2000 mg/l of kaolinite whereas 87% removal was achieved at a bentonite dose of 5657 mg/l.

10. An alum dose above 40 mg/l achieved an effluent total BOD_5 concentration less than 30 mg/l after 40 minutes settling. All doses of alum and bentonite tested reduced total BOD_5 concentration in the supernatant to less than 30 mg/l after three days of settling. Kaolinite did not show any ability to remove BOD.

However, kaolinite showed better ability than bentonite in removal of phosphates. Alum was the most effective in phosphorus removal. The concentration of TP was reduced, after three days of settling, from 5.5 mg/l to 2.6, 2.8 and 3.2 mg/l by alum, kaolinite and bentonite, respectively.

Generally, phosphorus was best removed at higher coagulant doses and longer settling times.

Alum, bentonite and kaolinite showed a decreasing order of removal of ammonia.

11. Crossflow microfiltration is a new technology which showed some potential and should be considered as one of the alternatives for algae removal. However, it requires more research than was possible in the course of this work.

12. A model was constructed to describe the changes in the concentration of phosphorus, as an indication of eutrophic status, in King Talal Reservoir in Jordan. The model indicated that the water of the reservoir will continue to deteriorate if the input load of phosphorus is maintained at the same increasing rate.

The model showed that reduction of phosphorus in Al-Samra effluent by new restrictions on phosphorus-based detergents, or even complete elimination of phosphorus from Al-Samra Wastewater Stabilization Ponds effluent, would not bring the quality of the lake to an oligotrophic state. This is because of the feed of phosphorus by other sources.

9.2 Recommendations

The following points summarize recommendations for further research:

1. A study is recommended to assess the performance and design of upflow rock filters for algae removal utilizing graded rock sizes.

2. It is recommended that a study be carried out on the effect of addition of small doses of different coagulants to the influent of the rock filter.

3. A long term study is required to determine the storage capacity and length of rock filter run.

4. Experiments on algae removal by the use of other low cost materials, such as volcanic ash, are recommended.

5. Crossflow microfiltration is a recent development which showed some potential for algae removal. Further research is recommended to define the effects of different variables on the process and to optimize its performance.

6. With respect to King Talal Reservoir, it is recommended that the model developed in this study be validated by continuous monitoring of the quality of the water behind the dam. This will be particularly important if it is required to predict the degree of reduction that should be applied to phosphorus input from diffuse sources. Accompanying this monitoring of the reservoir, a study is recommended to identify more precisely the contributions of different diffuse sources of P to the reservoir. Also, further study along Wadi Dhuleil and the Zarqa River is required to determine the sinks and sources of phosphorus along their courses and the degree of contribution of Al-Samra ponds' effluent to the phosphorus load to KTR.

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Table A–1: Rock filters operation results

SET No. 1
DATE 19/12/90
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.4	8.3	8.4	8.4	8.3	8.4	8.4	8.4	8.4	8.1	8.1	8.0	8.7
DO (mg/l)	3.8	1.4	2.0	1.7	1.1	2.1	1.2	1.8	1.9	0.6	1.5	0.6	1.2
TEMP. (°C)	13	10.7	10.9	10.9	11.7	10.3	11.0	11.4	12.3	19	21.3	22.3	20.2
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	420				87				70				50
TURBIDITY (NTU)	220	104	104	103	102	98	98	97	96	76	74	74	72
TSS(mg/l)	258	77	76	76	75	62	61	61	62	43	44	43	43
VSS (mg/l)	246				69				58				42
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 23/12/90
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.9	8.1	8.1	8.1	8.2	8.1	8.1	8.2	8.1	8.0	8.0	8.1	8.1
DO (mg/l)	1.3	1.1	1.1	1.3	1.2	1.1	1.0	1.0	1.1	1.4	1.0	1.0	1.2
TEMP. (°C)	15.1	15	15	15	15	15	15	15	15	16.4	16.4	16.4	16.4
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	387				92				59				37
TURBIDITY (NTU)	227	112	111	110	109	101	100	100	99	73	72	71	70
TSS(mg/l)	260	75	73	73	72	65	65	64	64	47	45	43	41
VSS (mg/l)	242				68				60				37
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 25/12/90
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1	8.2	8.3	8.2	8.2	8.3	8.3	8.3	8.2	8.1	8.0	8.0	8.2
DO (mg/l)	2.0	1.4	2.6	1.3	1.1	1.9	1.5	1.8	1.9	1.2	1.7	1.5	1.4
TEMP. (°C)	14	13.2	12.2	13.7	13.7	13.2	12.8	13.7	15.5	18.5	16.5	17.1	15.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	392				95				65				44
TURBIDITY (NTU)	224	112	112	108	108	104	102	101	100	70	70	69	68
TSS(mg/l)	255	74	73	70	70	68	65	63	63	38	37	36	35
VSS (mg/l)	238				66				59				33
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 31/12/90
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.4				8.4				8.4				8.3
DO (mg/l)	1.5				1.4				1.3				1.3
TEMP. (°C)	11.5				11.4				11.3				13.5
NH3 (mg/l)	3.5				3.3				3.4				3.2
TKN (mg/l)	20				16				15				12.5
P_total(mg/l)	6.5				6.4				6.3				6.0
PO4 (mg/l)	6.0				5.9				6.0				5.9
CHLOROPHYLL (µg/l)	362				93				70				40
TURBIDITY (NTU)	206	114	114	112	112	106	105	106	105	61	54	54	52
TSS(mg/l)	247	76	76	76	76	63	61	61	59	43	38	38	36
VSS (mg/l)	224				72				57				34
COD_total (mg/l)	400				271				248				202
COD_soluble (mg/l)	180				175				175				175
BOD_total (mg/l)	250				126				106				80
BOD_soluble (mg/l)	45				40				38				38

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 9/1/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.4				8.4				8.1				8.4
DO (mg/l)	0.9				1.2				0.8				1.3
TEMP. (°C)	11.0				11.0				11.1				16.3
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µmg/l	350				163				137				70
TURBIDITY (NTU)	196	103	104	104	102	97	96	98	98	76	74	74	72
TSS(mg/l)	219	147	147	146	146	137	136	134	134	107	103	101	99
VSS (mg/l)	209				139				128				94
COD total (mg/l)	435				360				350				325
COD soluble (mg/l)	190				183				182				182
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 12/1/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.3				8.3				8.2				8.3
DO (mg/l)	0.7				0.7				1.0				0.6
TEMP. (°C)	11.2				10.2				10.5				19.3
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	290				167				121				60
TURBIDITY (NTU)	143	112	111	110	109	101	100	100	99	73	72	71	70
TSS(mg/l)	172	160	155	152	152	151	145	142	136	107	104	103	101
VSS (mg/l)	166				148				134				100
COD total (mg/l)	400				365				360				350
COD soluble (mg/l)	173				166				166				160
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 15/1/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.4				8.4				8.3				8.1
DO (mg/l)	0.5				0.2				0.3				0.1
TEMP. (°C)	11.0				10.8				10.9				15.2
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	365				175				140				68
TURBIDITY (NTU)	220	112	112	108	108	104	102	101	100	70	70	69	68
TSS(mg/l)	250	138	138	137	138	141	$140	139	138	115	%107	104	101
VSS (mg/l)	244				136				136				99
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)	199				110				125				122
BOD_soluble (mg/l)	77				49				56				64

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 16/1/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.4				8.1				8.1				8.1
DO (mg/l)	0.5				0.6				0.5				0.7
TEMP. (°C)	12.1				11.3				11.1				18.2
NH3 (mg/l)	4				4				4				4
TKN (mg/l)	12				11.5				10.0				&9.7
P total(mg/l)	6.0				5.8				5.8				5.7
PO4 (mg/l)	5.7				5.6				5.6				5.6
CHLOROPHYLL (µg/l)													
TURBIDITY (NTU)													
TSS(mg/l)													
VSS (mg/l)													
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 20/1/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				7.7				7.8				7.9
DO (mg/l)	0 8				1.8				1.6				0.9
TEMP. (°C)	12.0				12.8				14.3				18.6
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	307				202				130				64
TURBIDITY (NTU)	173	134	125	121	120	115	113	111	110	88	80	79	77
TSS(mg/l)	202	159	154	150	146	140	140	140	140	120	114	111	110
VSS (mg/l)	196				144				135				107
COD_total (mg/l)	370				330				335				320
COD_soluble (mg/l)	180				167				173				153
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 21/1/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 02				8.23				8.27				8.18
DO (mg/l)	0 6				1.2				0.4				1.0
TEMP. (°C)	12.3				12 0				12.3				13.1
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µmg/l	290				187				120				56
TURBIDITY (NTU)	255	216	207	201	198	205	201	201	198	186	183	183	176
TSS(mg/l)	240	160	157	156	155	154	152	149	147	162	149	134	122
VSS (mg/l)	232				151				143				118
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 23/1/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				8.2				8.2				8.2
DO (mg/l)	0.7				0.7				0.7				0.6
TEMP. (°C)	10.3				10.2				10.2				11.4
NH3 (mg/l)	9.2				9.8				9.2				8.5
TKN (mg/l)	22.2				22.0				21.0				17.0
P total(mg/l)	5 5				5.4				5.4				5.3
PO4 (mg/l)	5 0				5.0				5.0				5.0
CHLOROPHYLL (µg/l)	363				248				168				80
TURBIDITY (NTU)	240	213	201	198	195	210	198	198	192	189	186	181	168
TSS(mg/l)	260	182	173	170	167	156	147	146	142	133	129	126	124
VSS (mg/l)	248				162				137				119
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 31/1/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1 5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8.1				8.0				8.1
DO (mg/l)	0.7				0.5				0.5				0.5
TEMP. (°C)	12 2				12.6				12.7				15.5
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	318				214				143				68
TURBIDITY (NTU)	209	148	139	139	116	124	118	117	114	99	94	90	86
TSS(mg/l)	231	171	169	167	165	150	148	148	145	120	116	112	108
VSS (mg/l)	220				160				141				105
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	235				217				193				170
BOD soluble (mg/l)	60				58				57				58

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 4/2/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.7				7.8				7.7
DO (mg/l)	0 6				0.7				0.6				0.6
TEMP. (°C)	11.6				10.7				11.0				17.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	234				89				65				33
TURBIDITY (NTU)	134	100	95	93	82	90	87	85	81	71	65	56	53
TSS(mg/l)	162	87	84	82	81	74	72	70	68	86	76	65	54
VSS (mg/l)	154	80	77	77	76	70	67	66	66	72	57	54	52
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	195				176				188				165
BOD soluble (mg/l)	95				95				95				95

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 6/2/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7 9				7.7				7.8				7.7
DO (mg/l)	0 5				0.3				0.5				0.4
TEMP. (°C)	10.7				10.5				10.6				19.5
NH3 (mg/l)	11.0				11.0				11.0				11.0
TKN (mg/l)	20				19 8				19.7				18
P total(mg/l)	5 6				5.5				5.4				5.2
PO4 (mg/l)	5.1				5.1				5.1				5.0
CHLOROPHYLL (µg/l)	265	81	81	80	79	60	57	58	58	46	40	37	34
TURBIDITY (NTU)	142	142	142	141	141	62	62	50	45	60	56	56	40
TSS(mg/l)	169	78	77	76	76	68	67	66	65	65	61	60	57
VSS (mg/l)	161				71				60				55
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 9/2/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.9				7.9				7.9				7.9
DO (mg/l)	0.5				0.4				0.5				0.5
TEMP. (°C)	9.8				9.0				8.8				20.1
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	233	78	77	77	77	66	66	66	64	30	26	26	25
TURBIDITY (NTU)	150	68	61	61	61	64	62	61	60	58	54	50	46
TSS(mg/l)	160	77	76	75	74	77	75	73	73	58	55	52	49
VSS (mg/l)	150				67				65				47
COD total (mg/l)	400				344				334				300
COD soluble (mg/l)	225				240				229				227
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 13/2/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.7				7.8				7.7				7.8
DO (mg/l)	0.3				0.2				0.4				0.5
TEMP. (°C)	10.3				10.3				10.3				11.9
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)	6.0				6.0				6.0				5.8
CHLOROPHYLL (µg/l)	227	85	84	83	82	67	65	63	61	38	35	33	31
TURBIDITY (NTU)	122	79	77	75	73	70	68	66	64	72	68	66	62
TSS(mg/l)	151	79	77	77	77	77	74	71	70	64	60	60	56
VSS (mg/l)	142				71				67				54
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 16/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.8				7.8				7.8
DO (mg/l)	1.0				1.1				0.9				1.0
TEMP. (°C)	11.5				11.5				11.5				13.2
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	86	53	52	52	52	53	51	51	50	38.5	35	32	31
TURBIDITY (NTU)	85	71	69	68	46	77	65	65	64	50	41	39	39
TSS(mg/l)	90	81	81	80	80	71	71	70	70	65	64	63	62
VSS (mg/l)	70				58				53				50
COD total (mg/l)	320				290				270				265
COD soluble (mg/l)	200				187				180				173
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 18/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.9				7.9				7.9
DO (mg/l)	0 9				0.9				1.0				0.8
TEMP. (°C)	11.7				10.9				10.8				21.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	201				93				79				36
TURBIDITY (NTU)	107	169	166	165	165	160	158	156	155	88	86	84	82
TSS(mg/l)	130	94	94	94	94	90	91	90	89	80	79	76	76
VSS (mg/l)	122				86				81				70
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	135				113				98				98
BOD soluble (mg/l)	55				65				46				60

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 20/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8.0				8.0				8.0
DO (mg/l)	0.7				0.7				0.7				0.4
TEMP. (°C)	19.1				18.3				18.5				15.4
NH3 (mg/l)	7				5.3				2.6				4.5
TKN (mg/l)	17.2				11.7				11.7				9.5
P total(mg/l)	6.0				5.9				5.9				5.7
PO4 (mg/l)	5 9				5.9				5.9				5.7
CHLOROPHYLL (µg/l)	201				102				74				35
TURBIDITY (NTU)	113	113	112	112	112	110	107	106	104	84	80	77	74
TSS(mg/l)	135	92	88	88	88	86	85	84	83	75	71	68	65
VSS (mg/l)	122				80				77				58
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 23/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.4				7.8				7.7				7.8
DO (mg/l)	0.7				0.5				0.5				0.5
TEMP. (°C)	13 0				13.7				14.0				23.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	140				77				61				30
TURBIDITY (NTU)	95	101	101	98	96	92	88	88	82	84	80	78	65
TSS(mg/l)	116	87	85	83	82	79	75	74	71	74	71	67	64
VSS (mg/l)	102				76				66				58
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 25/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.0				7.0				7.0				7.0
DO (mg/l)	0.7												
TEMP. (°C)	15 3				13.1				13.3				20.1
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	220	111	110	109	108	86	84	82	82	45	42	40	38
TURBIDITY (NTU)	128	123	118	116	112	116	116	112	105	74	69	66	50
TSS(mg/l)	152	83	83	83	84	84	82	82	80	55	53	52	50
VSS (mg/l)	142				77				76				48
COD_total (mg/l)	315				295				280				250
COD_soluble (mg/l)	210				210				210				210
BOD_total (mg/l)	138				123				114				90
BOD_soluble (mg/l)	84				70				68				60

Table A–1 contd: Rock filters operation results

SET No. 1
DATE 27/2/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.1				7.0				7.0				7.0
DO (mg/l)	0 6				0.9				0.8				0.9
TEMP. (°C)	15.7				13 3				13.8				20.2
NH3 (mg/l)	1 0				2.3				1.1				0.5
TKN (mg/l)	8 0				7.9				7.7				7.5
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	225				100				79				45
TURBIDITY (NTU)	131				109				101				60
TSS(mg/l)	156				86				82				56
VSS (mg/l)	137				77				74				52
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 2/3/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.9				7.8				7.7				7.9
DO (mg/l)	0 8				0.9				0.5				0.8
TEMP. (°C)	15.3				15.2				15.3				21.3
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	205	90	88	88	88	81	80	79	79	46	44	44	41
TURBIDITY (NTU)	95	69	68	68	68	65	64	63	63	51	51	50	50
TSS(mg/l)	122	90	90	90	90	88	86	85	84	60	57	58	58
VSS (mg/l)	116				85				80				56
COD total (mg/l)	380				280				275				230
COD soluble (mg/l)	187				180				170				156
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 4/3/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7 9				8 0				8.0				8.0
DO (mg/l)	0 8				0.3				0.3				0.3
TEMP. (°C)	14				12 0				12.7				17.7
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	270				109				89				45
TURBIDITY (NTU)	150	114	112	112	112	90	87	86	86	73	72	71	70
TSS(mg/l)	192	82	82	82	82	75	75	75	75	61	61	61	61
VSS (mg/l)	168				75				66				54
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	168				114				105				93
BOD soluble (mg/l)	96				72				70				66

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 6/3/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.9				7.7				7.9				7.8
DO (mg/l)	0 6				0.6				0.6				0.6
TEMP. (°C)	12.6				12.0				12.3				20.2
NH3 (mg/l)	8				7.8				7.9				7.6
TKN (mg/l)	20.1				19.4				17.3				15.7
P_total(mg/l)	6 5				6.1				6.0				3.2
PO4 (mg/l)	4 8				4.7				4.7				4.0
CHLOROPHYLL (µg/l)	207				92				78				39
TURBIDITY (NTU)	108	98	96	97	96	92	91	91	90	54	50	48	45
TSS(mg/l)	139	94	93	93	93	83	83	82	82	61	60	60	59
VSS (mg/l)	125				82				75				54
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 10/3/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8.1				7.9				7.9
DO (mg/l)	0.7				0 8				0.7				0.8
TEMP. (°C)	15 2				12 5				12.2				22.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	100				67				45				24
TURBIDITY (NTU)	76	73	70	70	69	65	65	64	64	40	35	32	31
TSS(mg/l)	102	82	81	79	78	74	71	70	68	50	43	40	38
VSS (mg/l)	95				72				64				36
COD_total (mg/l)	382				302				272				222
COD_soluble (mg/l)	257				203				190				170
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 11/3/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				8.0				8.0				8.0
DO (mg/l)	0.7				1.0				0.7				0.7
TEMP. (°C)	15.1				12.0				12.0				20.1
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	200				73				57				23
TURBIDITY (NTU)	106	94	89	87	85	72	70	65	62	55	47	44	44
TSS(mg/l)	138	74	73	73	72	57	54	52	51	38	33	29	28
VSS (mg/l)	128				68				47				28
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	138				120				128				111
BOD soluble (mg/l)	70				72				92				92

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 13/3/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				8.2				8.1				8.2
DO (mg/l)	0.8				1.0				1.0				1.0
TEMP. (°C)	16.8				12.8				13.1				20.5
NH3 (mg/l)	9.7				11.2				10.5				10.4
TKN (mg/l)	20.4				19.9				18.7				19.1
P total(mg/l)	6.3				6.0				6.0				5.7
PO4 (mg/l)	6.0				6.0				6.0				5.6
CHLOROPHYLL (µg/l)	198				65				58				35
TURBIDITY (NTU)	115	85	82	79	79	75	69	63	58	65	60	57	48
TSS(mg/l)	145	70	67	67	67	61	59	58	57	46	39	35	33
VSS (mg/l)	130				59				52				31
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 14/3/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.8				7.9				8.0
DO (mg/l)	0.8				0.8				0.8				0.5
TEMP. (°C)	14 9				14.3				14.5				21.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	205				64				55				30
TURBIDITY (NTU)	116	75	73	72	71	66	65	63	61	34	31	30	28
TSS(mg/l)	144	71	70	70	69	60	60	58	59	40	38	36	34
VSS (mg/l)	130				61				54				31
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 16/3/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8.1				8.1				8.1
DO (mg/l)	0 8				0.7				0.7				0.3
TEMP. (°C)	14 0				14 0				14.0				18.2
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	293				159				112				82
TURBIDITY (NTU)	140	116	116	116	112	116	116	114	108	84	76	74	68
TSS(mg/l)	200	118	118	118	118	108	108	108	107	85	83	81	80
VSS (mg/l)	178				110				95				77
COD_total (mg/l)	477				337				322				317
COD_soluble (mg/l)	253				220				210				206
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET NO. 2
DATE 18/3/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.9				8.0				8.1				8.0
DO (mg/l)	0 8				0.9				0.8				1.0
TEMP. (°C)	18.5				15.4				15.5				22.8
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	279				128				90				40
TURBIDITY (NTU)	184	110	109	108	104	101	100	99	98	91	90	89	88
TSS(mg/l)	228	122	122	121	122	110	110	109	109	82	82	81	81
VSS (mg/l)	218				106				96				72
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	200				161				140				130
BOD soluble (mg/l)	128				80				104				72

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 20/3/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.9				7.9				7.9
DO (mg/l)	0 8				0.5				0.5				0.6
TEMP. (°C)	16 0				13 3				13.1				17.1
NH3 (mg/l)	10 9				11.1				11.1				11.1
TKN (mg/l)	19 6				19.1				19.1				17.9
P total(mg/l)	4.1				3.7				3.4				3.0
PO4 (mg/l)	3 7				3.3				2.9				2.3
CHLOROPHYLL (µg/l)	283				136				95				45
TURBIDITY (NTU)	142	102	100	96	96	100	100	100	90	100	98	96	94
TSS(mg/l)	177	115	113	113	113	102	101	100	99	76	74	72	71
VSS (mg/l)	160				104				90				65
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 23/3/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.9				7.9				7.9
DO (mg/l)	0 8				0.8				0.8				0.8
TEMP. (°C)	14.5				12.1				11.7				15.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	233				79				62				32
TURBIDITY (NTU)	177	102	102	102	102	74	74	74	74	59	50	54	52
TSS(mg/l)	190	87	86	85	85	78	77	77	76	60	54	53	52
VSS (mg/l)	148				69				60				44
COD_total (mg/l)	470				340				310				290
COD_soluble (mg/l)	283				230				216				210
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 25/3/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.9				7.9				7.9
DO (mg/l)	0 9				0.8				0.8				0.8
TEMP. (°C)	15 0				12 8				12.6				20.1
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	201				99				73				37
TURBIDITY (NTU)	118	114	114	108	100	88	85	85	83	74	&70	66	62
TSS(mg/l)	138	93	92	91	91	83	82	81	80	66	64	64	64
VSS (mg/l)	120				78				68				54
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)	135				113				105				98
BOD_soluble (mg/l)	88				83				60				69

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 26/3/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8.1				7.9				8.1
DO (mg/l)	0 9				1.2				0.5				0.5
TEMP. (°C)	15.1				15 0				14.9				24.4
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	207				91				69				35
TURBIDITY (NTU)	112	75	75	75	75	67	67	66	666	63	62	62	62
TSS(mg/l)	136	92	92	92	92	76	75	74	73	62	62	61	60
VSS (mg/l)	120				80				63				52
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 31/3/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				8 1				8.1				8.1
DO (mg/l)	0 8				1 0				1.0				1.2
TEMP. (°C)	21				17 5				18				24.8
NH3 (mg/l)	6 32				9 4				5.8				6.8
TKN (mg/l)	15 0				19 6				17.9				16.5
P total(mg/l)	5 4				6 6				6.4				5.4
PO4 (mg/l)	4 4				5 8				5.9				5.2
CHLOROPHYLL (µg/l)	219				82				63				32
TURBIDITY (NTU)	125	97	95	94	94	87	86	85	84	60	57	56	56
TSS(mg/l)	154	90	86	84	84	81	80	80	79	64	58	58	58
VSS (mg/l)	132				71				69				50
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 1/4/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				8.1				8.1				8.1
DO (mg/l)	1.2				0.8				0.8				0.8
TEMP. (°C)	20.0				22				23				24
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	200				105				77				39
TURBIDITY (NTU)	140	100	98	98	96	90	88	88	86	60	58	56	54
TSS(mg/l)	150	90	89	89	89	71	71	71	71	65	62	59	58
VSS (mg/l)	124				83				59				50
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	140				103				99				70
BOD soluble (mg/l)	50				46				42				638

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 3/4/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				8.1				8.1				8.1
DO (mg/l)	1.5				0.8				0.8				0.8
TEMP. (°C)	21.8				22.5				23				27.6
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	240				80				64				32
TURBIDITY (NTU)	138	118	116	114	114	108	108	104	86	66	62	62	62
TSS(mg/l)	178	89	87	86	86	90	85	83	82	55	59	59	59
VSS (mg/l)	150				72				69				55
COD total (mg/l)	520				344				324				314
COD soluble (mg/l)	352				234				211				198
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 6/4/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				8.1				8.1				8.1
DO (mg/l)	1.5				1.2				1.2				1.4
TEMP. (°C)	19 0				15.3				15.1				26.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	281				142				105				50
TURBIDITY (NTU)	146	92	92	92	92	90	90	90	90	86	86	84	80
TSS(mg/l)	206	133	133	133	132	115	115	115	115	94	90	87	85
VSS (mg/l)	175				108				93				70
COD_total (mg/l)	526				376				370				346
COD_soluble (mg/l)	252				225				228				205
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET NO. 2
DATE 8/4/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				8.1				8.1				8.1
DO (mg/l)	1.7				1.5				1.5				1.5
TEMP. (°C)	18 7				18 5				18.7				26.0
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	289				127				103				61
TURBIDITY (NTU)	134	100	99	98	98	90	88	88	87	72	68	65	62
TSS(mg/l)	218	128	127	127	127	119	119	118	118	86	84	82	82
VSS (mg/l)	182				101				98				72
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)	168				115				110				83
BOD_soluble (mg/l)	71				71				64				64

Table A–1 contd: Rock filters operation results

SET No. 2
DATE 10/4/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				8.3				8.3				8.1
DO (mg/l)	1.0				1.2				1.2				1.2
TEMP. (°C)	17.4				13.4				13.0				18.6
NH3 (mg/l)	10 9				10 9				11.0				11.1
TKN (mg/l)	22				21.5				21.5				18.0
P total(mg/l)	5 0				5 8				5.7				5.4
PO4 (mg/l)	4 5				5.4				5.3				5.3
CHLOROPHYLL (µg/l)	300				129				105				52
TURBIDITY (NTU)	144	108	106	106	106	86	84	82	80	87	84	81	78
TSS(mg/l)	222	138	137	136	136	126	124	124	124	91	90	89	87
VSS (mg/l)	189				118				106				75
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 4/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7 6				7.5				7.4				7.4
DO (mg/l)	0 4				0.4				0.3				0.5
TEMP. (°C)	17 6				15.4				17.5				20.5
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	222				90				82				40
TURBIDITY (NTU)	152	104	100	98	98	97	693	93	93	73	670	68	64
TSS(mg/l)	174	116	111	110	110	108	105	104	104	75	71	70	70
VSS (mg/l)	142				85				78				56
COD total (mg/l)	620				360				320				300
COD soluble (mg/l)	295				255				240				225
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 6/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.5				7.5				7.4
DO (mg/l)	0.2				0.4				0.3				0.3
TEMP. (°C)	21.3				18.4				20.8				24.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	381	128	127	126	125	120	116	112	108	55	50	47	43
TURBIDITY (NTU)	178	122	122	120	120	135	133	132	133	80	78	78	76
TSS(mg/l)	232	98	95	95	95	118	116	115	115	76	76	76	72
VSS (mg/l)	198				77				90				70
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	195				90				88				82
BOD soluble (mg/l)	92				70				66				60

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 8/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				7.7				7.7				7.7
DO (mg/l)	0.2				0.2				0.2				0.2
TEMP. (°C)	22.3				18.9				21.0				24.6
NH3 (mg/l)	0.2				0.1				0.1				0.05
TKN (mg/l)	16.8				21				17.4				17
P total(mg/l)	5.5				4.4				3.7				4.2
PO4 (mg/l)	4.5				3.1				1.9				2.6
CHLOROPHYLL (µg/l)	381				120				105				50
TURBIDITY (NTU)	198	116	108	100	95	87	79	77	74	74	66	62	62
TSS(mg/l)	236	104	100	101	100	87	85	85	84	76	72	68	68
VSS (mg/l)	197				86				72				57
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 11/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.9				7.9				7.9
DO (mg/l)	0 2				0.1				0.1				0.1
TEMP. (°C)	23 2				20.3				23.1				25.7
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	335	125	113	108	108	112	102	96	90	65	57	56	55
TURBIDITY (NTU)	176	111	113	108	108	85	84	83	83	66	64	62	60
TSS(mg/l)	210	113	109	107	106	93	90	89	90	75	70	71	68
VSS (mg/l)	182				87				76				63
COD_total (mg/l)	750				330				320				300
COD_soluble (mg/l)	440				226				213				206
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 13/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7 5				7.8				7.7
DO (mg/l)	0 5				0.1				0.1				0.1
TEMP. (°C)	23 2				20 2				23.2				25.1
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (mg/l)	435				130				103				59
TURBIDITY (NTU)	311	184	181	178	178	168	164	164	160	118	115	112	110
TSS(mg/l)	400	105	101	100	101	95	93	92	90	70	70	69	70
VSS (mg/l)	308				82				70				60
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)	260				142				135				127
BOD_soluble (mg/l)	99				92				80				80

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 15/5/91
HYDRAULIC LOADING 1.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				7.8				7.9				7.9
DO (mg/l)	0 6				0.3				0.2				0.4
TEMP. (°C)	23 3				18.4				19.7				22.5
NH3 (mg/l)	0.1				0.1				0.1				0.1
TKN (mg/l)	17.0				14.3				14.2				14.1
P total(mg/l)	11.7				11.3				11.0				9.8
PO4 (mg/l)	5 0				6.0				6.0				4.7
CHLOROPHYLL (µg/l)	160				99				86				50
TURBIDITY (NTU)	156	140	140	140	140	106	104	102	100	90	90	90	90
TSS(mg/l)	182	138	135	135	135	115	104	98	94	79	71	70	70
VSS (mg/l)	146				108				75				61
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 18/5/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				7.7				7.7				7.8
DO (mg/l)	0 6				0.1				0.1				0.3
TEMP. (°C)	23 5				20 0				20.5				25.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	185	82	80	78	77	71	69	69	67	43	39	34	32
TURBIDITY (NTU)	117	78	78	78	78	71	70	70	68	73	69	66	64
TSS(mg/l)	136	90	90	90	88	82	80	78	75	86	78	71	66
VSS (mg/l)	106				75				56				52
COD total (mg/l)	430				320				310				270
COD soluble (mg/l)	290				253				240				206
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 20/5/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				8.1				8.2				8.0
DO (mg/l)	0 8				0.3				0.3				0.4
TEMP. (°C)	23 6				21.1				22.2				27.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	138	73	72	72	72	62	60	58	56	29	29	29	28
TURBIDITY (NTU)	152	90	85	85	83	80	75	71	71	71	70	64	62
TSS(mg/l)	180	86	86	85	86	78	76	74	72	74	69	64	62
VSS (mg/l)	135				70				56				49
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	171				105				112				82
BOD soluble (mg/l)	95				80				57				62

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 22/5/91
HYDRAULIC LOADING 0.5 m3/m3 d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.7				7.7				7.7
DO (mg/l)	1 2				0 3				0.3				0.3
TEMP. (°C)	24				22.1				23.6				24.0
NH3 (mg/l)	0 2				2.1				2.4				3.7
TKN (mg/l)	18 2				18.2				17.7				17.2
P total(mg/l)	5 8				5				5.2				4.0
PO4 (mg/l)	1 7				2 9				3.7				1.9
CHLOROPHYLL (µg/l)	200	84	80	78	78	60	60	60	58	29	29	27	21
TURBIDITY (NTU)	204	88	84	83	82	88	81	77	77	72	70	68	67
TSS(mg/l)	245	89	83	84	84	88	85	83	81	76	72	69	66
VSS (mg/l)	196				65				62				50
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 26/5/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				8.0				8.0				8.0
DO (mg/l)	1.5				0.4				0.3				0.4
TEMP. (°C)	24.1				21.3				21.3				24.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	170	70	70	70	70	55	54	53	52	32	31	30	29
TURBIDITY (NTU)	180	110	108	106	106	100	94	91	90	76	76	74	74
TSS(mg/l)	210	93	93	92	92	80	78	77	77	67	64	61	58
VSS (mg/l)	160				70				59				45
COD total (mg/l)	470				310				270				220
COD soluble (mg/l)	210				226				193				190
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 27/5/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				8 0				7.8				7.7
DO (mg/l)	1 5				0.4				0.9				0.9
TEMP. (°C)	23 8				20.4				19.8				23.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	133	74	74	74	74	67	66	64	62	37	34	32	30
TURBIDITY (NTU)	130	108	108	106	98	70	66	66	66	43	42	41	38
TSS(mg/l)	144	82	82	80	81	78	77	78	78	66	60	57	54
VSS (mg/l)	112				63				62				46
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	128				75				75				60
BOD soluble (mg/l)	52				62				48				52

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 29/5/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				8.2				8.0				7.7
DO (mg/l)	1.6				1.2				1.0				0.9
TEMP. (°C)	21.7				18.1				17.6				21.0
NH3 (mg/l)	0.4				1.7				1.7				2.3
TKN (mg/l)	14.6				16.7				15.8				16.7
P total(mg/l)	5.0				4.9				5.1				4.2
PO4 (mg/l)	1.6				3.3				3.6				2.5
CHLOROPHYLL (µg/l)	123	68	66	66	66	58	56	54	54	32	32	30	30
TURBIDITY (NTU)	120	60	60	58	58	56	52	52	50	32	32	30	30
TSS(mg/l)	145	90	86	85	84	71	70	70	70	73	66	60	58
VSS (mg/l)	116				67				56				46
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 1/6/91
HYDRAULIC LOADING 0.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				7.7				7.7				7.9
DO (mg/l)	1.8				1.0				1.0				1.0
TEMP. (°C)	22.0				19.1				22.0				24.4
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µmg/l)	120	70	70	68	68	54	54	49	49	28	26	24	17
TURBIDITY (NTU)	99	88	86	85	86	85	82	79	81	62	58	52	47
TSS(mg/l)	122	95	95	94	94	94	91	87	85	66	62	57	54
VSS (mg/l)	94				69				65				42
COD total (mg/l)	470				410				340				260
COD soluble (mg/l)	170				206				167				153
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 3/6/91
HYDRAULIC LOADING 0 5 m3/m3 d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				8.1				8.1				8.1
DO (mg/l)	1.7				0.9				0.9				0.9
TEMP. (°C)	23 5				20.5				23.4				25.5
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	125	68	68	68	68	56	56	54	54	28	28	26	26
TURBIDITY (NTU)	142	86	86	84	84	82	80	78	73	60	53	53	48
TSS(mg/l)	166	94	94	94	93	83	83	83	83	62	57	52	52
VSS (mg/l)	133				76				66				41
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 5/6/91
HYDRAULIC LOADING 0 5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				8 1				8.1				8.1
DO (mg/l)	1 8				1.0				0.9				0.9
TEMP. (°C)	23 7				20				23.4				25.0
NH3 (mg/l)	0 4				0 4				1.0				1.0
TKN (mg/l)	13 1				13 6				15.5				14.1
P total(mg/l)	5 2				4 9				4.7				4.4
PO4 (mg/l)	2 7				3 7				3 0				3.3
CHLOROPHYLL (µg/l)	153	72	72	72	72	49	49	47	45	24	22	20	19
TURBIDITY (NTU)	142	90	91	91	90	73	69	68	62	61	59	57	56
TSS(mg/l)	170	97	93	91	92	80	77	77	70	60	52	50	48
VSS (mg/l)	136				75				57				38
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 11/6/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 3				8 0				8.0				8.0
DO (mg/l)	1 6				1.5				1.7				1.6
TEMP. (°C)	25 3				23 0				24.9				27.1
NH3 (mg/l)	0 5				0.5				0.5				0.5
TKN (mg/l)	9 6				9.5				9.4				9.3
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	129	102	102	102	102	62	58	58	56	37	34	33	32
TURBIDITY (NTU)	164	117	117	117	114	130	118	115	110	80	72	72	66
TSS(mg/l)	190	143	136	130	129	127	120	113	107	85	82	82	82
VSS (mg/l)	148				98				77				64
COD total (mg/l)	440				344				300				250
COD soluble (mg/l)	254				279				203				169
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 12/6/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				7 9				7.9				7.9
DO (mg/l)	1 4				0 8				0.7				0.7
TEMP. (°C)	23 5				23 2				24.8				27.0
NH3 (mg/l)	0 5				0 5				0.5				0.5
TKN (mg/l)	15 8				15 3				15.0				14.6
P total(mg/l)	5 0				4 9				4.8				4.7
PO4 (mg/l)	1 4				2 9				3.0				0.9
CHLOROPHYLL (µg/l)	112	142	140	140	136			62	60	46	44	42	40
TURBIDITY (NTU)	164	130	128	126	122	134	132	128	124	126	120	120	120
TSS(mg/l)	191	143	140	138	138	155	129	123	121	103	99	97	97
VSS (mg/l)	154				111				98				78
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 15/6/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.1				7.8				7.9				7.9
DO (mg/l)	1 0				0 6				0.7				0.6
TEMP. (°C)	25 3				23.1				24.9				28.1
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	175	146	144	142	142	107	103	100	97	44	40	36	35
TURBIDITY (NTU)	215	162	140	126	122	126	120	120	97	97	91	91	87
TSS(mg/l)	234	188	170	153	137	151	126	126	117	110	104	100	97
VSS (mg/l)	192				110				95				78
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	154				85				80				68
BOD soluble (mg/l)	68				55				52				41

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 17/6/91
HYDRAULIC LOADING 2.0 m3/m3 d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				8.1				8.1				8.0
DO (mg/l)	1 0				0 5				0.5				0.5
TEMP. (°C)	25 5				22 4				25.4				28.0
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	163	169	152	137	135	125	115	114	96	51	48	48	47
TURBIDITY (NTU)	180	120	120	116	114	120	115	108	108	90	81	77	73
TSS(mg/l)	202	142	138	134	130	113	110	109	107				80
VSS (mg/l)	156				100				84				58
COD total (mg/l)	410				430				400				280
COD soluble (mg/l)	244				280				247				207
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 19/6/91
HYDRAULIC LOADING 2.0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				8 0				8.0				7.9
DO (mg/l)	1 0				0.8				0.8				0.8
TEMP. (°C)	25 5				22.4				25.0				28.0
NH3 (mg/l)	0 3				0.4				0.4				0.2
TKN (mg/l)	18 9				15.3				13.6				13.6
P total(mg/l)	5 4				5 2				5.0				4.2
PO4 (mg/l)	3.1				2.9				3.9				3.5
CHLOROPHYLL (µg/l)	225				145				105	55	51	49	50
TURBIDITY (NTU)	162	108	108	106	106	162	162	160	146	96	88	92	85
TSS(mg/l)	185	129	125	122	121	149	142	136	130	97	93	91	91
VSS (mg/l)	145				96				102				71
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 26/6/91
HYDRAULIC LOADING 2 0 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 1				7 9				7.9				7.9
DO (mg/l)	1.1				0 9				0.9				0.9
TEMP. (°C)	26 8				23 0				24.0				27.0
NH3 (mg/l)	0 4				0.2				0 2				0.1
TKN (mg/l)	16 0				15 3				14.8				14.3
P total(mg/l)	5 7				4 0				3.8				3.6
PO4 (mg/l)	5 0				2 7				4.7				3.5
CHLOROPHYLL (µg/l)	355	280	164	164	162	171	170	171	170	61	60	61	60
TURBIDITY (NTU)	152	57	54	53	53	89	84	79	74	40	38	37	34
TSS(mg/l)	179	136	136	136	135	150	137	134	133	85	82	88	87
VSS (mg/l)	140				107				107				69
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 29/6/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 8	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH –	8.1				8 0				8.1				8.1
DO (mg/l)	1.1				1.2				0.8				0.8
TEMP. (°C)	27.1				24.7				25.5				27.5
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	227	105	98	96	91	57	54	52	50	46	44	42	40
TURBIDITY (NTU)	175	105	101	101	100	66	60	58	57	41	39	637	35
TSS(mg/l)	136	92	88	85	85	70	59	59	55	43	40	38	36
VSS (mg/l)	108				67				43				29
COD total (mg/l)	540				290				190				180
COD soluble (mg/l)	255				203				161				150
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 1/7/91
HYDRAULIC LOADING 0.1 m3/m3 d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 2				8.1				8.1				8.1
DO (mg/l)	0 8				1.0				1.0				1.0
TEMP. (°C)	25				22				23				26
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	398	104	103	99	95	86	75	74	70	43	32	28	26
TURBIDITY (NTU)	180	75	72	70	68	58	55	53	53	46	40	34	30
TSS(mg/l)	194	80	77	77	77	69	64	60	58	45	40	38	37
VSS (mg/l)	160				62				47				32
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	171				53				45				53
BOD soluble (mg/l)	49				51				37				46

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 3/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7 9	7.6			7.6	7.6			7.6	7.6			7.7
DO (mg/l)	2 5	0 4			0.7	0.7			0.3	1.0			1.5
TEMP. (°C)	31 9	28			28.1	28 5			30.0	31.5			34
NH3 (mg/l)	1.3				0 9				0.5				0.8
TKN (mg/l)	16 0				14 9				14.7				12.3
P_total(mg/l)	5 6				4 5				5.7				4.1
PO4 (mg/l)	3 2				4 0				4.9				3.0
CHLOROPHYLL (µg/l)	387	96	96	96	95	63	55	55	52	32	29	27	26
TURBIDITY (NTU)	142	75	73	67	67	66	60	56	56	30	25	23	22
TSS(mg/l)	168	71	70	69	68	68	60	60	57	39	37	36	35
VSS (mg/l)	134				56				47				30
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 6/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.7				7.7				7.8
DO (mg/l)	2 3				0 8				0.8				1.6
TEMP. (°C)	32				28				30				34
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	270	81	78	78	74	84	77	71	66	55	52	52	50
TURBIDITY (NTU)	90	68	66	64	62	82	73	67	62	31	30	29	28
TSS(mg/l)	110	76	72	72	70	91	83	75	69	42	38	36	35
VSS (mg/l)	90				57				56				30
COD_total (mg/l)	400				190				150				143
COD_soluble (mg/l)	254				153				127				133
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 8/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.7				7.7				7.7
DO (mg/l)	2 4				0 8				0.8				0.8
TEMP. (°C)	30 0				27				29				34
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	372	85	83	81	80	89	80	72	69	49	47	46	46
TURBIDITY (NTU)	114	78	75	74	70	45	45	44	44	32	30	29	28
TSS(mg/l)	154	84	82	80	79	67	68	68	68	42	40	40	41
VSS (mg/l)	124				67								35
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	98				47				40				47
BOD soluble (mg/l)	44				37				32				42

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 10/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER ≈10cm				ROCK DIAMETER ≈5 cm				ROCK DIAMETER ≈1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1 0	1 5	2 0	0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7 7				7.7				7.7				7.7
DO (mg/l)	3 0				0 8				0.8				0.8
TEMP. (°C)	31				28				29				34
NH3 (mg/l)	1 8				1 4				1.4				&1.1
TKN (mg/l)	15 5				13 8				12.9				11.4
P total(mg/l)	5 4				5 9				5.0				3.4
PO4 (mg/l)	4 3				5 4				4.1				2.1
CHLOROPHYLL (µg/l)	408	106	102	102	101	102	85	77	73	64	53	50	47
TURBIDITY (NTU)	188	82	82	80	80	81	78	77	76	35	33	32	31
TSS(mg/l)	218	78	75	78	74	77	72	72	73	39	38	38	37
VSS (mg/l)	174				61				60				30
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 14/7/91
HYDRAULIC LOADING 0.1 m3/m3 d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.7				7.7				7.6				7.7
DO (mg/l)	3.2				0.9				1.2				1.1
TEMP. (°C)	31.5				28				30				34.3
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	344	89	85	82	80	81	77	76	73	76	72	61	50
TURBIDITY (NTU)	210	86	78	76	76	73	65	64	64	42	42	42	40
TSS(mg/l)	240	82	80	78	76	80	70	64	62	40	38	38	38
VSS (mg/l)	178				62				51				31
COD total (mg/l)	470				240				234				220
COD soluble (mg/l)	190				169				160				155
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 15/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.7				8.0				7.9				7.9
DO (mg/l)	4.0				0.8				0.7				0.7
TEMP. (°C)	31.3				28.2				29.8				34.1
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	355	80	75	75	73	79	70	70	70	80	73	47	42
TURBIDITY (NTU)	150	103	102	100	100	98	90	85	80	37	35	33	30
TSS(mg/l)	168	75	73	71	70	70	63	64	64	40	38	37	33
VSS (mg/l)	144				60				50				31
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	135				87				66				60
BOD soluble (mg/l)	80				74				51				54

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 17/7/91
HYDRAULIC LOADING 0.1 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.5				7.5				7.5
DO (mg/l)	4.0				1.0				1.0				1.0
TEMP. (°C)	30.1				27.2				28.7				33.7
NH3 (mg/l)	4.0				0.04				0.2				0.04
TKN (mg/l)	16.2				15.5				15.5				12.9
P total(mg/l)	5.6				5.9				5.3				2.1
PO4 (mg/l)	3.7				5.4				4.8				1.7
CHLOROPHYLL (µg/l)	452	104	98	94	90	85	71	70	66	73	69	67	55
TURBIDITY (NTU)	152	100	90	84	81	70	60	55	52	67	60	55	51
TSS(mg/l)	178	88	83	83	80	63	59	58	57	48	43	42	40
VSS (mg/l)	142				65				46				32
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 20/7/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				8.0				8.0				8.0
DO (mg/l)	2.8				2.0				2.0				2.0
TEMP. (°C)	30.8				27.7				29.4				33.7
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	434	198	198	196	196	172	168	168	154	83	77	77	73
TURBIDITY (NTU)	151	141	130	125	125	124	112	102	99	74	71	71	70
TSS(mg/l)	202	187	187	186	150	160	153	149	147	112	102	94	90
VSS (mg/l)	164				125				125				71
COD total (mg/l)	606				271				276				191
COD soluble (mg/l)	278				173				167				127
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 22/7/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.7				7.7				8.2
DO (mg/l)	1.0				1.1				1.1				1.1
TEMP. (°C)	31				28.1				29.6				34
NH3 (mg/l)													
TKN (mg/l)													
P_total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	452	193	193	189	191	150	150	147	147	107	101	101	96
TURBIDITY (NTU)	150	131	122	122	121	122	120	119	118	91	88	85	82
TSS(mg/l)	222	190	181	179	178	184	184	180	180	102	98	95	93
VSS (mg/l)	170				160				158				78
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)	180				60				73				47
BOD_soluble (mg/l)	33				23				42				23

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 24/7/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				8.5				7.5				7.5
DO (mg/l)	0.7				0.5				0.5				0.5
TEMP. (°C)	31				28				29.6				34
NH3 (mg/l)	4.9				4.3				4.4				4.6
TKN (mg/l)	16.7				16.0				14.8				15.5
P_total(mg/l)	4.4				4.0				3.7				2.1
PO4 (mg/l)	3.6				2.7				2.1				0.9
CHLOROPHYLL (µg/l)	400	211	200	192	192	164	144	142	140	86	80	77	73
TURBIDITY (NTU)	178	123	123	122	122	118	116	114	115	112	102	96	92
TSS(mg/l)	202	142	141	141	141	132	130	130	130	130	112	106	103
VSS (mg/l)	160				114				102				81
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 27/7/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.7				7.7				7.8
DO (mg/l)	0.7				0.5				0.5				0.5
TEMP. (°C)	31.5				28				29.6				34.1
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	408	244	222	214	190	172	165	160	155	108	97	86	86
TURBIDITY (NTU)	282	170	156	150	146	136	136	136	130	134	126	124	118
TSS(mg/l)	336	169	168	167	167	163	163	163	162	140	130	125	124
VSS (mg/l)	300				150				148				116
COD total (mg/l)	660				460				334				290
COD soluble (mg/l)	240				235				156				167
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 29/7/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	7.8				7.8				7.8				7.8
DO (mg/l)	0.7				0.6				0.6				0.6
TEMP. (°C)	31				28				30				33.8
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (mg/l)	408	194	194	194	183	189	185	174	168	110	106	103	97
TURBIDITY (NTU)	272	145	143	144	138	140	136	134	120	96	86	82	78
TSS(mg/l)	324	162	162	162	160	160	155	152	150	116	106	99	94
VSS (mg/l)	260				124				120				75
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	195				87				93				60
BOD soluble (mg/l)	27				42				51				28

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 3/8/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7.8				7.8				7.8
DO (mg/l)	1.0				0 8				0.7				0.9
TEMP. (°C)	31				28				30				34
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	436	271	247	237	230	192	187	184	182	80	87	85	83
TURBIDITY (NTU)	270	155	151	150	146	140	130	126	124	104	96	92	90
TSS(mg/l)	324	163	162	161	160	157	155	154	154	95	92	90	90
VSS (mg/l)	245				125				120				72
COD total (mg/l)	525				306				210				180
COD soluble (mg/l)	245				213				180				137
BOD total (mg/l)													
BOD soluble (mg/l)													

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 5/8/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0 5	1.0	1.5	2 0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8 0				7 8				7.8				7.9
DO (mg/l)	1 0				0.7				0.7				0.7
TEMP. (°C)	31 2				28 0				30				34
NH3 (mg/l)													
TKN (mg/l)													
P total(mg/l)													
PO4 (mg/l)													
CHLOROPHYLL (µg/l)	393	264	258	236	230	203	190	182	182	93	80	75	75
TURBIDITY (NTU)	274	165	160	160	152	160	150	140	130	160	146	134	128
TSS(mg/l)	324	127	125	125	125	124	120	121	119	108	102	100	98
VSS (mg/l)	252				97				92				78
COD total (mg/l)													
COD soluble (mg/l)													
BOD total (mg/l)	189				105				87				73
BOD soluble (mg/l)	71				64				46				42

Table A–1 contd: Rock filters operation results

SET No. 3
DATE 7/8/91
HYDRAULIC LOADING 1.5 m3/m3.d

PARAMETER	INFLUENT	ROCK DIAMETER =10cm				ROCK DIAMETER =5 cm				ROCK DIAMETER =1 cm			
		HEIGHT (m)				HEIGHT (m)				HEIGHT (m)			
		0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
pH	8.0				8.0				7.8				7.8
DO (mg/l)	1.0				0.7				0.7				0.7
TEMP. (°C)	31.2				28.1				28				34
NH3 (mg/l)	5.1				4.8				3.9				3.4
TKN (mg/l)	18.2				17.7				16.7				16.0
P_total(mg/l)	4.9				4.2				4.9				3.9
PO4 (mg/l)	2.1				4.0				3.7				1.3
CHLOROPHYLL (µg/l)	413	240	240	240	240	215	200	200	188	112	97	95	93
TURBIDITY (NTU)	215	140	136	133	130	124	118	116	112	122	122	110	106
TSS(mg/l)	240	128	125	124	123	118	117	116	115	95	93	93	91
VSS (mg/l)	200				104				97				77
COD_total (mg/l)													
COD_soluble (mg/l)													
BOD_total (mg/l)													
BOD_soluble (mg/l)													

APPENDIX - B

Solution of Equation 2.10

The change of concentration with respect to depth was presented in Eq. 2.6:

$$\frac{\partial c}{\partial x} = -\lambda c \quad \ldots\ldots (2.6)$$

The filter coefficient, Λ, was expressed in Eq. 2.10. And simply using the symbols of the dimensionless groups I, $1/P_e$, S, $1/R_e$ that represent mechanisms of interception, diffusion, gravity settling and hydrodynamic action respectively, Eq. 2.10 is rewritten as follows:

$$\Lambda = (I)^{\alpha}\left(\frac{1}{P_e}\right)^{\beta}(S)^{\gamma}\left(\frac{1}{R_e}\right)^{\delta} \quad \ldots\ldots (2.10)$$

Using the following assumptions, the average values of these dimensionless groups were calculated in Table (B-1) for experimental sets 1 and 2 (see Section 6.1.2):

d_p = 12 μm for set 1; 15 μm for set 2
T = 12.4°C for set 1; 16°C for set 2
μ = 1.227×10^{-3} N.s/m² for set 1; 1.112 N.s/m² for set 2

The values of λ at each hydraulic loading for sets 1 and 2 were calculated in Table (B-2) by solving Eq. 2.6 as follows:

$$\ln\frac{c}{c_o} = -\lambda x$$

Appendix-B

Taking x=2 m, C_o influent concentration and C_2 effluent concentration gives:

$$\lambda = -\frac{1}{2}\ln\frac{C_2}{C_{in}} \quad \ldots\ldots\ldots (C\text{-}1)$$

Linearizing Eq. 2.10 by taking the logarithms of both sides, results in Eq. B-2.

$$\text{Log}(A) = \text{Log const.} + \alpha\ \text{Log}(I) + \beta\ \text{Log}\ (1/P_e) + \gamma\ \text{Log}\ (S) + \delta\ \text{Log}(1/R_e) \ldots\ (B\text{-}2)$$

As an example of formulating Eq. B-2 to solve for α, β, γ, and δ, an hydraulic loading 0.1m³/m².d (v=0.2m/d) for the filter of rock 1 cm diameter was taken. In this filter the following data are applicable, from Tables (B-1) and (B-2):

λ = 0.928 , Λ=0.00928, Log(Λ) = -.0324

I = 1.2×10^{-3}, Log(I) = -2.921

$1/P_e$ = 1.26×10^{-7}, Log $(1/P_e)$ =-6.900

S = $8.179\times10_{-1}$, Log (S) =-0.087

$1/R_e$ = 53 , Log$(1/R_e)$ = 1.724

Thus, the first equation was formulated as follows:

$$(-2.0324) = \text{Log const.} + (-2.921)(\alpha) + (-6.900)(\beta) + (-0.087)(\gamma) + (1.724)(\delta)$$

The rest of the equations were formulated in the same way, as follows:

$$(-2.2807) = \text{Log const.} + (-2.921)(\alpha) + (-7.599)(\beta) + (-0.786)(\gamma) + (1.025)(\delta)$$

$$(-2.3994) = \text{Log const.} + (-2.921)(\alpha) + (-7.900)(\beta) + (-1.087)(\gamma) + (o.724)(\delta)$$

$$(-2.3891) = \text{Log const.} + (-2.921((\alpha) + (-8.076)(\beta) + (-1.388)(\gamma) + (0.548((\delta)$$

Appendix-B

(-2.5241) = Log const. +(-2.921)(α)+(-8.201)(β)+(-1.388)(γ)+(0.423)(δ)

(-2.1066) = Log const. +(-2.824)(α)+(-6.954)(β)+(0.149)(γ)+(1.682)(δ)

(-2.2439) = Log const. +(-2.824)(α)+(-7.653)(β)+(-0.550)(γ)+(0.983)(δ)

(-2.3275) = Log const. +(-2.824)(α)+(-7.954)(β)+(-0.851)(γ)+(0.682)(δ)

(-2.3992) = Log const. +(-2.824)(α)+(-8.130)(β)+(-1.027)(γ)+(0.506)(δ)

(-2.4432) = Log const. + (-2.824)(α)+(-8.255)(β)+(-1.152)(γ)+(0.380)(δ)

(-1.4602) = Log const. +(-3.620)(α)+(-7.600)(β)+(-0.032)(γ)+(1.025)(δ)

(-1.6988) = Log const. +(-3.620)(α)+(-8.300)(β)+(-0.731)(γ)+(0.326)(δ)

(-1.8542) = Log const. +(-3.620)(α)+(-8.600)(β)+(-1.032)(γ)+(0.025)(δ)

(-1.8925) = Log const. +(-3.620)(α)+(-8.775)(β)+(-1.208)(γ)+(-0.151)(δ)

(-2.0160) = Log const. +(-3.620)(α)+(-8.900)(β)+(-1.333)(γ)+(-0.276)(δ)

(-1.6039) = Log const. +(-3.523((α)+(-7.653)(β)+(0.204)(γ)+(0.983)(δ)

(-1.6636) = Log const. +(-3.523)(α)+(-8.352)(β)+(-0.495)(γ)+(0.284)(δ)

(-1.7974) = Log const. +(-3.523)(α)+(-8.653)(β)+(-0.796)(γ)+(-0.017)(δ)

(-1.4292) = Log const. +(-3.523)(α)+(-8.829)(β)+(-0.971)(γ)+(-0.193)(δ)

(-1.9840) = Log const. +(-3.523)(α)+(-8.954)(β)+(-1.097)(γ)+(-0.318)(δ)

(-1.2148) = Log const. +(-3.921)(α)+(-7.900)(β)+(7.321x10^{-3})(γ)+(0.724)(δ)

(-1.4602) = Log const. +(-3.921)(α)+(-8.599)(β)+(-0.692)(γ)+(0.0253)(δ)

(-1.6227) = Log const. +(-3.921)(α)+(-8.900)(β)+(-0.993)(γ)+(-0.276)(δ)

(-1.6518) = Log const. +(-3.921)(α)+(-9.076)(β)+(-1.169)(γ)+(-0.452)(δ)

(-1.8421) = Log const. +(-3.921)(α)+(-9.201)(β)+(-1.294)(γ)+(-0.577)(δ)

(-1.3988) = Log const. +(-3.824)(α)+(-7.954)(β)+(0.244)(γ)+(0.018700)(δ)

(-1.4353) = Log const. +(-3.824)(α)+(-8.653)(β)+(-0.455)(γ)+(-0.680)(δ)

(-1.5664) = Log const. +(-3.824)(α)+(-8.954)(β)+(-0.756)(γ)+(-0.981)(δ)

(-1.7494) = Log const. +(-3.824)(α)+(-9.130)(β)+(-0.932)(γ)+(-1.157)(δ)

(-1.8224) = Log const. +(-3.824)(α)+(-9.255)(β)+(-1.057)(γ)+(-1.282)(δ)

APPENDIX – B

Table B–1: Average values of groups for different removal mechanisms

SET	GROUP	GROUP VALUE FOR SHOWN APPROACH VELOCITIES				
		ROCK DIAMETER 1cm, POROSITY 0.37				
		0.2 m/d	1.0 m/d	2.0 m/d	3.0 m/d	4.0 m/d
1	I	1.200E–03	1.200E–03	1.200E–03	1.200E–03	1.200E–03
1	1/Pe	1.260E–07	2.519E–08	1.260E–08	8.397E–09	6.298E–09
1	S	8.179E–01	1.636E–01	8.179E–02	5.452E–02	4.089E–02
1	1/Re	5.300E+01	1.060E+01	5.300E+00	3.533E+00	2.650E+00
2	I	1.500E–03	1.500E–03	1.500E–03	1.500E–03	1.500E–03
2	1/Pe	1.112E–07	2.224E–08	1.112E–08	7.413E–09	5.560E–09
2	S	1.410E+00	2.820E–01	1.410E–01	9.400E–02	7.050E–02
2	1/Re	4.804E+01	9.608E+00	4.804E+00	3.203E+00	2.402E+00

Table B–1 ctd: Average values of groups for different removal mechanisms

SET	GROUP	GROUP VALUES FOR SHOWN APPROACH VELOCITIES				
		ROCK DIAMETER 5cm, POROSITY 0.42				
		0.2 m/d	1.0 m/d	2.0 m/d	3.0 m/d	4.0 m/d
1	I	2.400E–04	2.400E–04	2.400E–04	2.400E–04	2.400E–04
1	1/Pe	2.519E–09	5.038E–09	2.519E–09	1.679E–09	1.260E–09
1	S	9.284E–01	1.857E–01	9.284E–02	6.189E–02	4.642E–02
1	1/Re	1.060E+01	2.120E+00	1.060E+00	7.067E–01	5.300E–01
2	I	3.000E–04	3.000E–04	3.000E–04	3.000E–04	3.000E–04
2	1/Pe	2.224E+08	4.448E–09	2.224E–09	1.483E–09	1.112E–09
2	S	1.601E+00	3.201E–01	1.601E–01	1.068E–01	8.003E–02
2	1/Re	9.608E+00	1.922E+00	9.608E–01	6.405E–01	4.804E–01

Table B–1 ctd: Average values of groups for different removal mechanisms

SET	GROUP	GROUP VALUES FOR SHOWN APPROACH VELOCITIES				
		ROCK DIAMETER 10 cm, POROSITY 0.46				
		0.2 m/d	1.0 m/d	2.0 m/d	3.0 m/d	4.0 m/d
1	I	1.200E–04	1.200E–04	1.200E–04	1.200E–04	1.200E–04
1	1/Pe	1.260E–08	2.519E–09	1.260E–09	8.397E–10	6.298E–10
1	S	1.017E+00	2.034E–01	1.017E–01	6.779E–02	5.084E–02
1	1/Re	5.300E+00	1.060E+00	5.300E–01	3.533E–01	2.650E–01
2	I	1.500E–04	1.500E–04	1.500E–04	1.500E–04	1.500E–04
2	1/Pe	1.112E–08	2.224E–09	1.112E–09	7.413E–10	5.560E–10
2	S	1.753E+00	3.506E–01	1.753E–01	1.169E–01	8.762E–02
2	1/Re	1.044E+00	2.089E–01	1.044E–01	6.962E–02	5.222E–02

Table B–2: Calculation of filter coefficient for experimental sets 1 and 2

SET	v (m/d)	TSS IN (mg/l)	AVERAGE EFFLUENT TSS (mg/l)			VALUES OF FILTER COEFFICIENT		
			rock 10cm	rock 5cm	rock 1cm	rock 10cm	rock 5cm	rock 1cm
1	0.2	248	73.25	62	38.75	0.609775	0.693147	0.928149
2	0.2	159	71.5	58.75	33.25	0.399603	0.497806	0.782424
1	1.0	154	77	69	54	0.346573	0.401423	0.523984
2	1.0	183	87.83	76.83	58.5	0.367041	0.433945	0.570229
1	2.0	138	85.67	79.17	62.17	0.238375	0.277828	0.39869
2	2.0	152	88.33	80.33	59.33	0.2714	0.318868	0.470382
1	3.0	227	145.33	136	100.33	0.222971	0.256147	0.408242
2	3.0	188	131.67	119	84.67	0.178071	0.228659	0.39884
1	4.0	211	158.25	143.5	116	0.143841	0.192761	0.299134
2	4.0	159	117.67	105	77.33	0.15051	0.207471	0.360411

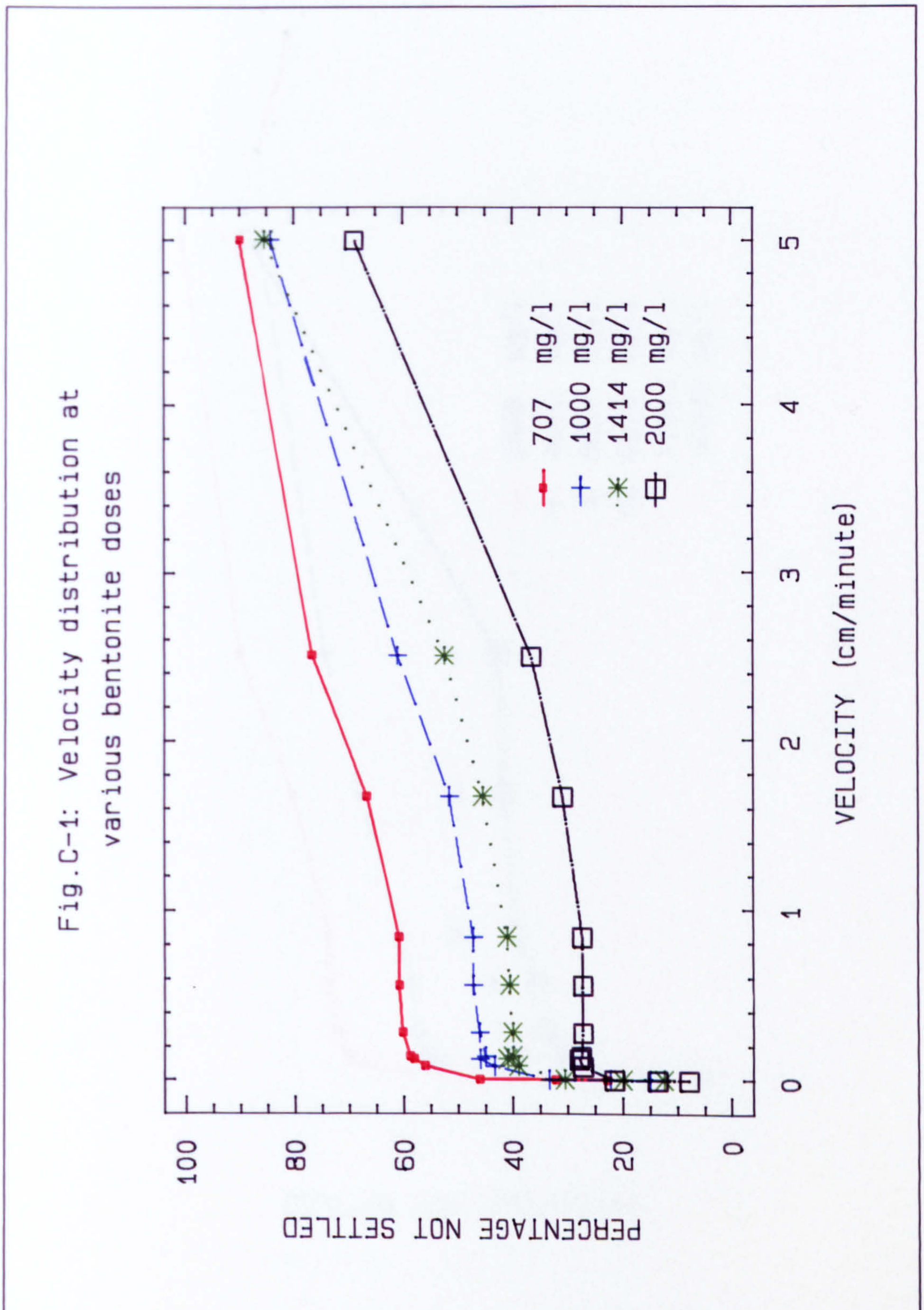


Fig.C-1: Velocity distribution at various bentonite doses

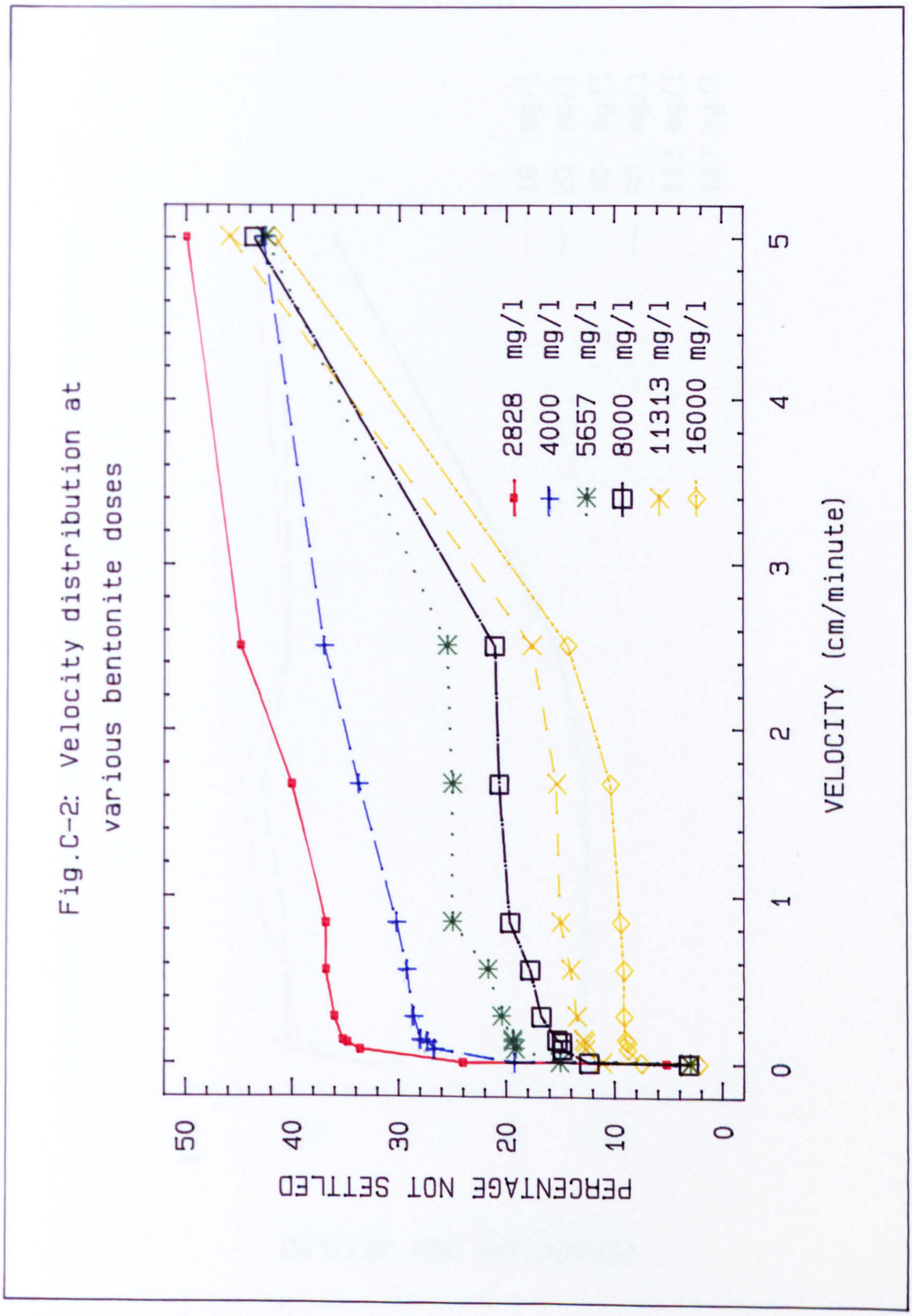
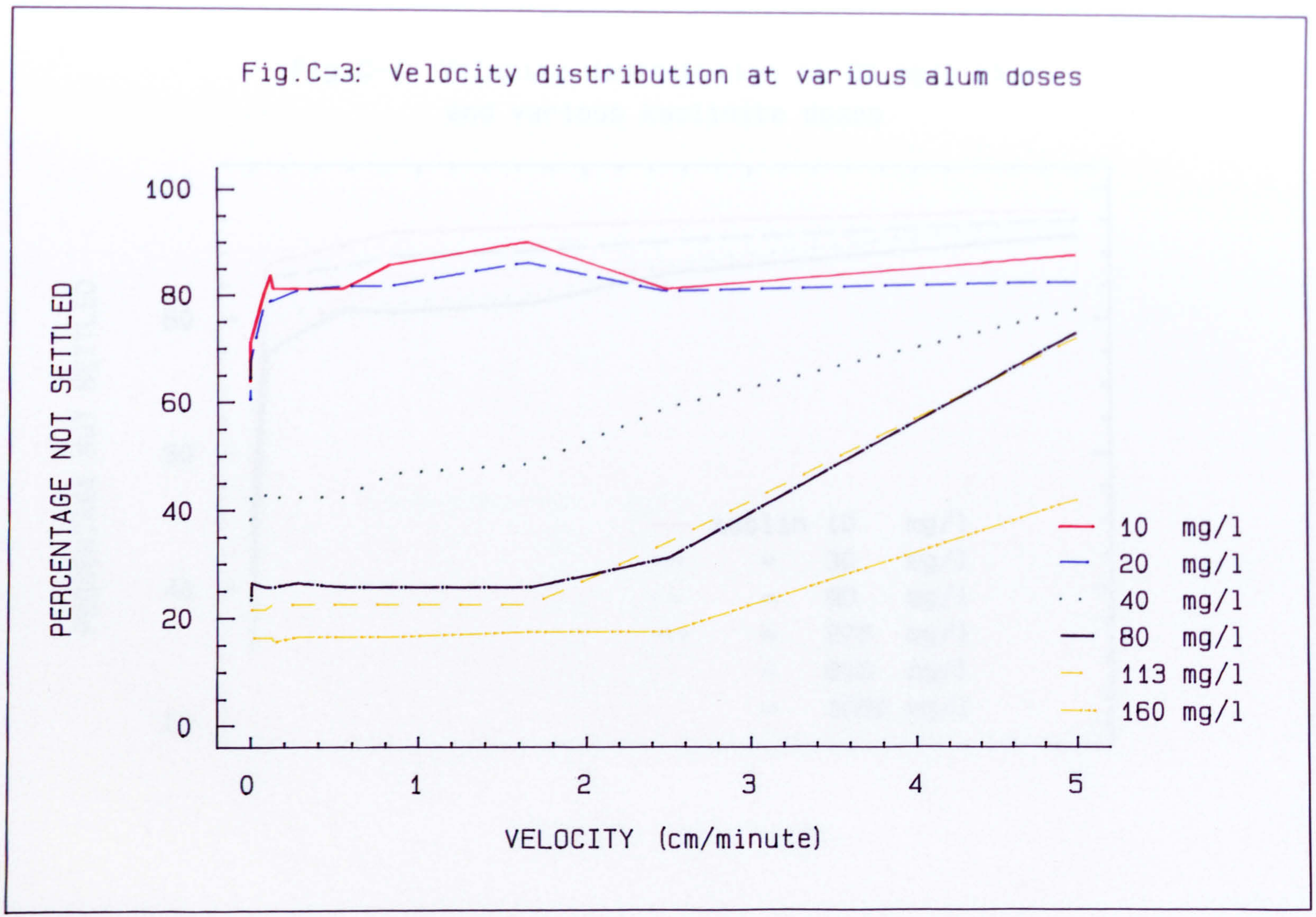


Fig.C-2: Velocity distribution at various bentonite doses

Fig.C-3: Velocity distribution at various alum doses



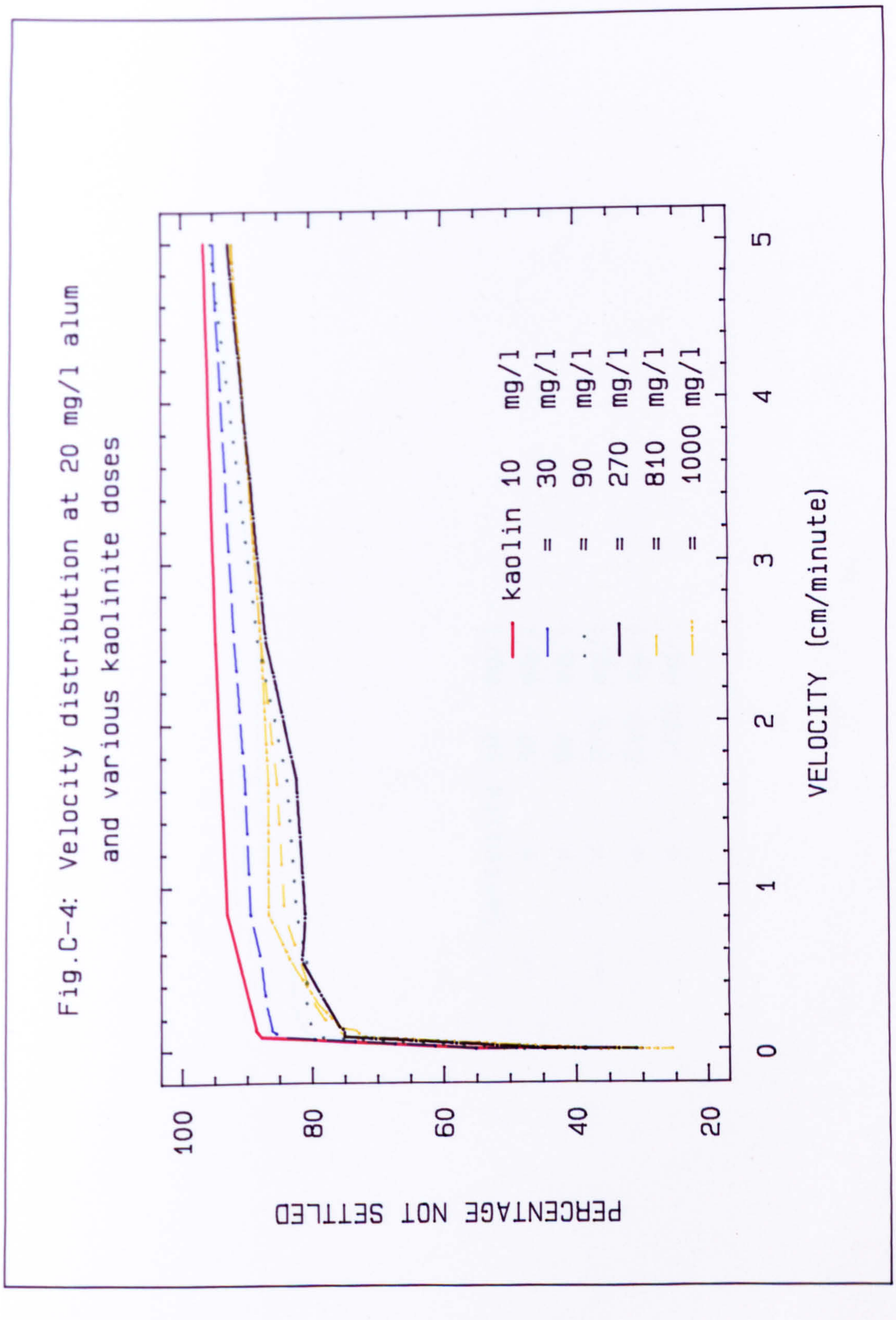


Fig.C-4: Velocity distribution at 20 mg/l alum
and various kaolinite doses
kaolin 10 mg/l
= 30 mg/l
= 90 mg/l
= 270 mg/l
= 810 mg/l
= 1000 mg/l
VELOCITY (cm/minute)
0
1
2
3
4
5
PERCENTAGE NOT SETTLED
100
80
60
40
20

Fig.C-5: Velocity distribution at 20 mg/l alum and various bentonite doses

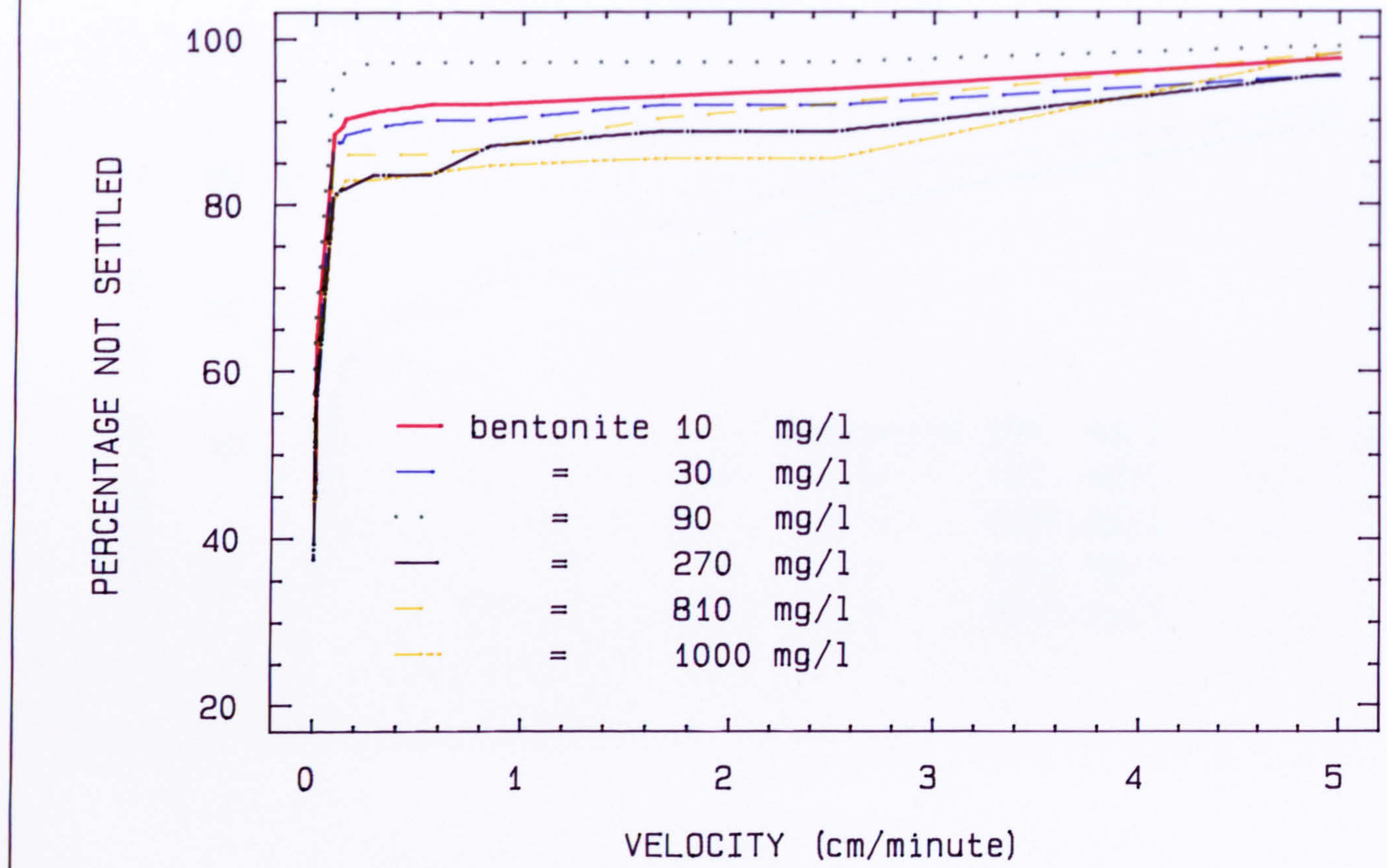


Fig.C-6: Velocity distribution at 1500 mg/l kaolinite and various bentonite doses

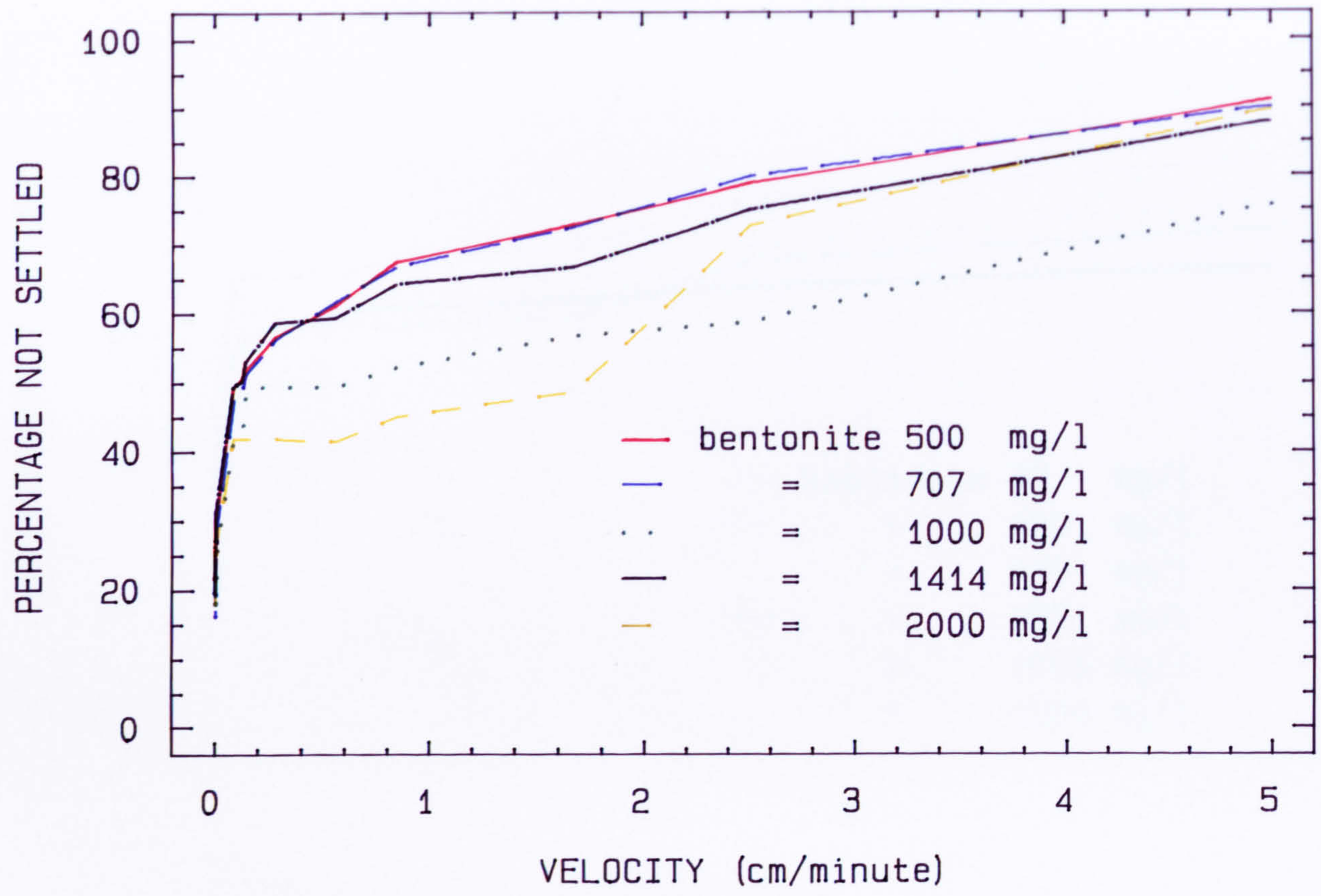


Fig.C-7: Velocity distribution at 4000 mg/l bentonite and various kaolinite doses